# THE

# RESIDUAL STRENGTH OF SOILS

# AND LANDSLIDE STABILITY

A thesis

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To my father

## ABSTRACT

This thesis studies the influence of the residual shear strength of soils on the stability of landslides.

The effect of rate of shear on the residual strength of soils is studied in a modified Bromhead ring shear apparatus. It is found that the loss of soil about the perimeter of the apparatus may introduce pore pressures at fast shearing rates, and a method of deriving the maximum shearing rate is proposed.

A silty clay is tested over a wide range of shearing rates and normal stresses. With increased rate of shearing, the soil shows a reduction in strength at slow rates, and an increase at fast rates. These results are compared with studies of rate effects for other soils and rocks, and a mechanism to explain the observed phenomena is proposed.

The behaviour of four landslides is reviewed in the light of the residual strength study, and it is concluded that rate effects are one of the important factors in determining the future stability of a landslide.

## ACKNOWLEDGMENTS

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# NOTATION

| | |
|---|---|
| $A$ | annular cross sectional area of the sample |
| $a,b$ | coefficients in transient constitutive model |
| $c_p$ | cohesion at peak strength |
| $d$ | depth of sample |
| $d_p$, $d_u$, $d_l$ | depth of platen, upper sample, and lower sample |
| $d_r$ | sample degradation rate |
| $F_1,F_2$ | loading arm forces |
| $F_f$ | frictional force between shaft and bush |
| $G$ | specific gravity of the solid particles |
| $g$ | acceleration of gravity |
| $h_{up}$, $h_{lp}$ | head loss across upper and lower platen |
| $h_{us}$, $h_{ls}$ | head loss across upper and lower samples |
| $k_o$ | coefficient of lateral earth pressure |
| $k_s$, $k_p$ | soil and platen permeability |
| $L_i$ | characterisitic length |
| $l$ | depth of lip |
| $M_f$ | moment caused by friction between shaft and bush |
| $M_{f3}$ | moment error due to soil friction in gap |
| $M_s$ | mass of solids |
| $P$ | normal load from loading yoke |
| $Q_D$ | water demand |
| $r_i$, $r_o$ | inner and outer radius of upper platens |
| $r_s$ | radius of shaft |
| $r_{tb}$ | radius of thrust bearing |
| $T_c$ | characterisitic time |

| | |
|---|---|
| $T_c^{Bishop}$ | characteristic time in Bishop apparatus |
| $T_c^{Bromhead}$ | characteristic time in Bromhead apparatus |
| t | time |
| u | pore water pressure |
| V | volume of soil |
| $V_w$ | volume of water |
| v | rate of shearing |
| $v_{max}$ | maximum shearing rate |
| $v_{max}^{Bishop}$ | maximum shearing rate in Bishop apparatus |
| $v_{max}^{Bromhead}$ | maximum shearing rate in Bromhead apparatus |
| $v_*$ | standard rate of shearing |
| $v_u$ , $v_l$ | pore fluid flux velocities to the upper and lower platens. |
| w | moisture content |
| $w_i$, $w_c$ | initial and critical state moisture contents |
| ß | viscosity |
| $ß_a$ | absolute viscosity |
| $\delta\ \tau_f$ | net effect of friction on measured shear strength |
| $\delta\ \tau_{f1}$ | change in measured shear strength due to shaft friction |
| $\delta\ \tau_{f2}$ | change in measured shear strength due to friction moment |
| $\delta\ \tau_{f3}$ | error in the shear stress as a consequence of soil friction in gap |
| $\delta\ \tau_{f4}$ | change in shear stress due to friction in thrust bearing |
| $\dot{\epsilon}$ | strain rate |
| $\dot{\epsilon}^*$ | standard strain rate |
| $\mu_b$ | coefficient of friction between shaft and bush |
| $\mu_{tb}$ | coefficient of friciton for the thrust bearing |

| | |
|---|---|
| $\mu_*$ | coefficient of friction at standard shearing rate |
| $\rho_w$ | density of water |
| $\xi$ | error in measured residual shear strength |
| $\sigma$ | total normal stress |
| $\sigma_y$ | tensile strength of steel in yield |
| $\sigma_y^*$ | tensile strength of steel in yield at standard strain rate |
| $\sigma'$ | effective stress |
| $\tau$ | shear strength of soil |
| $\tau_y$ | yield stress in steel |
| $\Omega$ | transient variable in constitutive model |
| $\varnothing_p$ | peak friction angle |
| $\varnothing_r$ | residual friction angle |

# CHAPTER 1

# INTRODUCTION

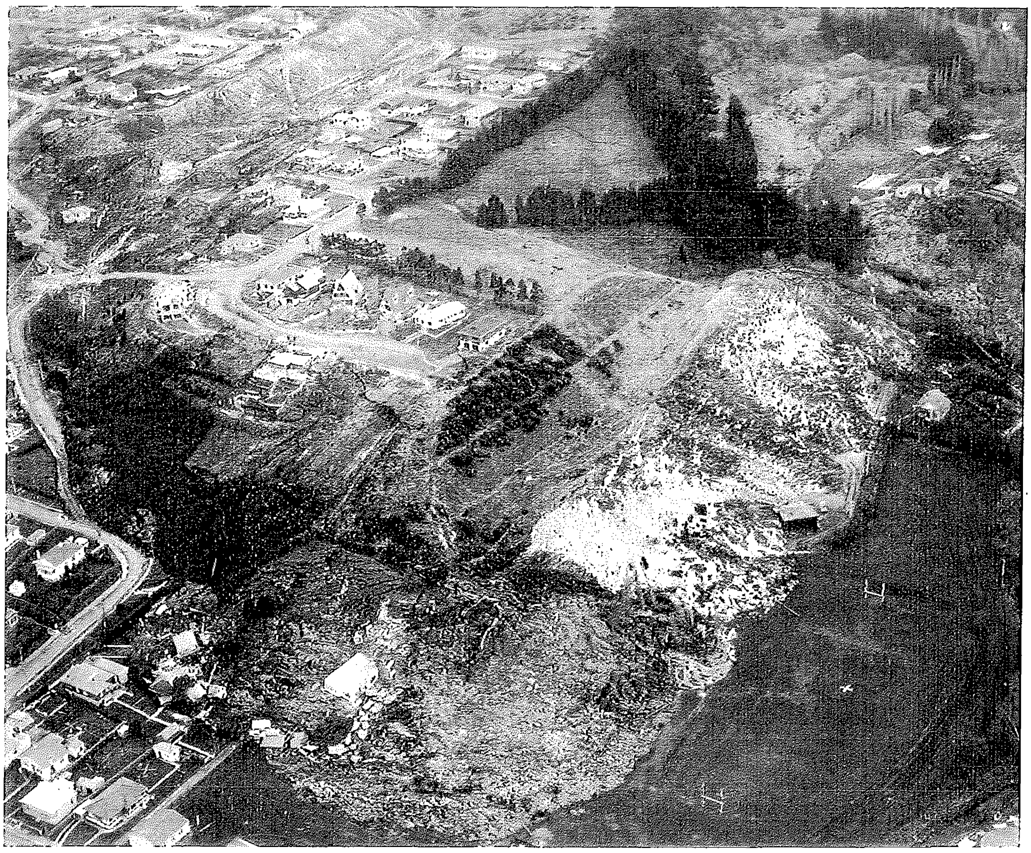
## 1.0 AIM OF RESEARCH

New Zealand's geological diversity and youth has produced landslides of considerable cost to mankind. The urban development of East Abbotsford, in Dunedin, was disrupted and subsequently abandoned as a result of a landslide in 1979. The Ruahihi Hydro Electric Power Project in the Bay of Plenty suffered a severe setback in 1981 when a landslide destroyed part of the canal. Most recently, the completion of the Clyde Dam in Central Otago has been delayed three years and stabilization works costing $400 million have been initiated out of concern about the stability of creeping slopes on the perimeter of the reservoir. These civil engineering works and many other projects throughout New Zealand have been disrupted by landslides putting lives at risk and costing millions of dollars.

The stability of landslides is going to be of increasing importance for New Zealand in the decades ahead. Similar problems to those at Clyde affect the Roxborough and Hawea reservoirs and the proposed development of the Lower Clutha River. Population growth in the major cities is encouraging the development of steeper and more landslide prone terrain.

International experience also suggests that an increasing proportion of fatalities during earthquakes and floods are caused by landslides. These factors suggest that geotechnical engineers are going to be confronted with an increasing number of difficult decisions regarding the risk posed by landslides.

The aim of this research project is to enhance the understanding of landslide behaviour so as to reduce the risk to life and to minimise costs.



Photograph 1.1 - East Abbotsford Landslide - August 1979

## 1.1 BACKGROUND TO LANDSLIDE STABILITY

Karl Terzaghi observed :

> ..if a landslide comes as a surprise to the eyewitness, it would be more accurate to say that the observers failed to detect the phenomena which preceded the slide.
> Terzaghi (1950) p. 110.

Despite this significant statement forty years ago, the mechanisms of landslide behaviour are poorly understood. Sophisticated analytical methods have been developed to predict the static factor of safety for slopes of every possible shape and description. However, very little is available to the practising engineer in assessing the long-term stability of creeping slopes.

Typically, a slope may creep for months, decades or centuries without rapid failure. The movement is usually less than 30mm/year and often occurs seasonally. In response to an earthquake, a rainfall event or some human activity the slope may accelerate and become a landslide as illustrated in Figure 1.1, reproduced from Terzaghi (1950). The key issues to be determined by the practising engineer are whether a landslide will occur, and if so when, and what actions may be taken to prevent it.

Empirical approaches have been attempted by some researchers. Saito (1980) analyzed the data from many Japanese landslides in developing an empirical methodology for predicting the

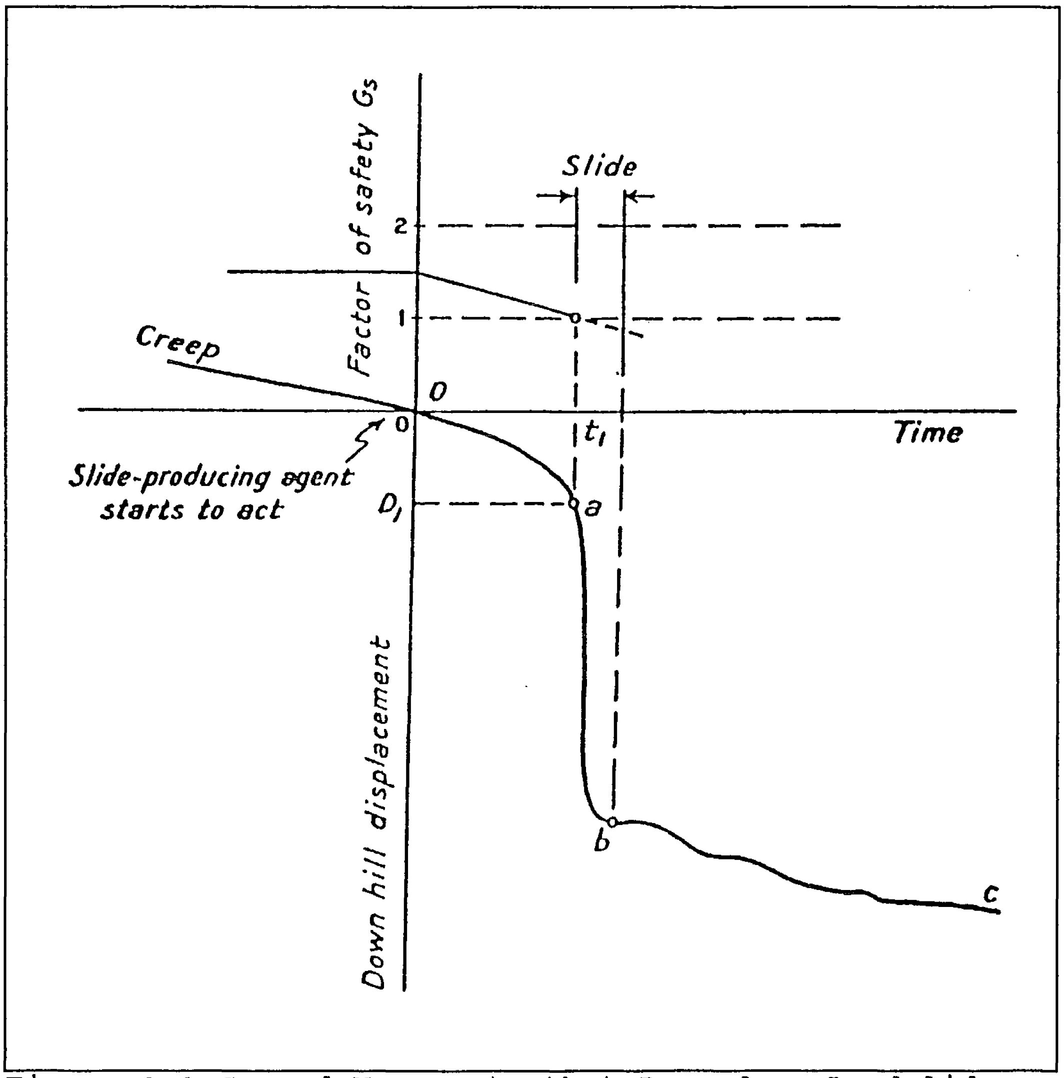



Figure 1.1 Ground Movements that Precede a Landslide
Terzaghi (1950) p. 111.

time to failure of a landslide. Using logarithmic graph paper and plotting landslide displacements and times, the method enabled the prediction of failure. In practice the method is inadequate in that the predictions only become possible within hours of failure. The method takes into account neither the chronological events that may stabilise or destabilise the slide, nor factors, such as soil type and slope geometry, which are known to play a significant role in the failure mechanism.

A more theoretical approach is required that takes into account factors like soil type, pore pressure changes and loading variations. Such factors are accounted for in traditional slope analysis methods but these methods ignore any dynamic factors affecting stability. A knowledge of the dynamic characteristics of the landslide is necessary to bridge the gap between current slope analysis methods and the problem of creeping slopes.

The critical unknown in understanding the dynamic behaviour of the landslide is the relationship between the residual shear strength of the soil and the rate of shearing. In simplified theories, the residual shear strength of a soil is independent of the rate of shearing and the slope remains completely static until the strength is exceeded and failure occurs. The fact that this does not describe actual slope behaviour suggests that there is a weak relationship between the residual shear strength and the rate of shearing. This was indirectly acknowledged by Terzaghi when he produced Figure 1.1, in which the slope is shown to accelerate before the factor of safety becomes unity.

The nature of this relationship could be investigated either in the field or laboratory. Both methods have advantages and disadvantages. A field test could be conducted by monitoring the movement of an existing landslide in response to changing loads. This method has advantages in that the soil is in an undisturbed state and problems of scale are eliminated.

However, the logistics of loading and measuring an in-the-field landslide would be difficult and expensive. Furthermore, the exact geology and geometry of the slide would not be known. A laboratory investigation is far more flexible and exact. However questions about the effects of using a disturbed sample of soil and the distortions created by the testing machine must be carefully addressed. The laboratory investigation of the residual shear strength in this thesis examines in depth these testing difficulties.

In this investigation it is important to recognise that stability can be affected by what might normally be disregarded as insignificant factors. Previous research indicates that the residual shear strength does not vary by more than 5% in response to changes in the rate of shearing. In stable slopes, it is reasonable to dispense with these phenomena as insignificant, but in a creeping slope small changes in soil strength or loading may have a major impact on stability. A knowledge of these internal landslide mechanisms is critical to understanding the stability of a creeping slope.

The research in this thesis may have important implications for some first-time landslides. In the case of a first-time slide initiated by an earthquake, the internal processes under scrutiny in this thesis will have a significant influence on whether such a landslide re-stabilises or accelerates to catastrophic failure.

## 1.2 FORMAT OF THESIS

Laboratory investigations into the rate effects on residual shear strength require the use of the ring shear apparatus. The testing specifications of this research project demanded significant modifications to the ring shear machine used. Chapter 2 of this thesis describes these modifications and outlines the system for acquiring data from the apparatus.

Chapter 3 examines a significant laboratory problem in conducting ring shear tests that may inadvertently distort measurements of the residual shear strength. Since the problem is not avoidable, an estimation of the magnitude of distortion is made.

Chapter 4 analyses the ring shear and direct shear results contained in Appendix III and IV. It describes the testing procedure and discusses the variation of the residual shear strength in relation to displacement and normal stress. The observed results in the experimental work studying the effect of shearing rate on residual strength are analysed.

Chapter 5 compares the conclusions of the rate effect study with laboratory investigations of other researchers. These include comparisons with other soils as well as those for rock, artificial soils and steel. The chapter draws general conclusions from these results about the nature of rate effects.

Chapter 6 considers the implications of the issues addressed in the research project and recommends future research directions.

A list of all references appears at the end of the thesis, followed by detailed description of soil testing in the appendices.

# CHAPTER 2

# THE RING SHEAR APPARATUS

## 2.0 INTRODUCTION

The ring shear apparatus has become recognised as the most appropriate method of studying the post-peak strength behaviour of soils and, particularly, for the determination of the residual shear strength (Skempton, 1985). This is because the ring shear apparatus is able to subject the soil sample to very large shearing displacements. This is necessary in geomechanics problems that involve assessing the stability of structures where the soil has undergone significant displacements.

Landslides are one example of this type of problem. Typically, the soil in the shear zone of a landslide will deform significantly prior to rapid failure. The East Abbotsford landslide had crept 3 m prior to failure. (Coombes & Norris, 1981). Similarly the Jizukiyama slide had displaced 2 m and the Ruahihi slope failure had sheared 0.5 m prior to final collapse (Salt, 1988b). These examples illustrate the necessity of being able to subject a laboratory sample of soil to shearing displacements of several metres.

Neither triaxial nor direct shear testing equipment provide satisfactory results at large shearing displacements. The cylindrical shape of the triaxial apparatus makes shear measurements at displacements of more than a few millimetres impossible. Large displacements in the direct shear apparatus are possible only by repeatedly reversing the direction of shear. Uncertainties as to the effect of the reversals make this method less than satisfactory.

A detailed knowledge of the ring shear apparatus is an essential component of this research project. To achieve the principal aim of identifying the effects of the rate of shearing on residual strength, very accurate measurements are required. Understanding how these measurements may be distorted by machine effects is critical to the reliability of this study's conclusions. This chapter reviews the historical development of the various types of ring shear apparatus, details the design of the machine used in this project, and describes the various problems encountered in testing.

## 2.1 LITERATURE REVIEW

The first known rotary ring shear apparatus was designed to determine the bearing strength of foundations (A.S.C.E., 1917). This was prior to the development of modern soil mechanics testing equipment. The apparatus twisted a cylindrically shaped sample of soil under a normal load (See Figure 2.1a). The cylindrical geometry of the sample meant that the shearing stresses and volume changes decreased to zero at the centre of the sample. This serious inadequacy was probably the reason the apparatus failed to find favour.

The concept of shearing the sample on a ring shaped failure plane was first advanced by Tiederman in 1933. While the sample in his apparatus was cylindrical, only a ring shaped plate in the base rotated (See Figure 2.1b). The non-uniform shear strain distribution made it difficult to determine the normal stresses on the failure surface, and the apparatus was not successful for this reason (Tiederman 1937).

The idea of using a ring shaped sample was developed independently by several researchers in the late 1930's. The most successful of these designs was that developed by Juul Hvorslev. His design involved shearing a confined ring shaped sample of soil under constant normal load (See Figure 2.1c). The apparatus was stress-controlled and could be used to shear the sample at mid-depth or just below the top platen. The ultimate shearing resistance after failure was found to be the same for both cases. Hvorslev concluded that the preferred test method was to shear the sample just below the top platen, as this minimised soil losses (Hvorslev, 1939).

Interest in the ring shear apparatus waned for a period of approximately 25 years during which the significance of the minimum shearing resistance to practical engineering problems was underestimated. In 1971 a combined research project by Imperial College London and the Norwegian Geotechnical Institute developed a complex ring shear apparatus (Bishop et al, 1971). This has subsequently been used by many researchers in the study of residual strength and will be referred to as the Bishop apparatus.

The major progress in design of the Bishop apparatus was the introduction of a mechanism to control the size of the gap between the upper and lower confining rings (See Figure 2.1d). The design also differed from Hvorslev's machine in that the lower platen was rotated rather than the upper and the Bishop device was strain, not stress, controlled.

Bishop's design proved well suited to research but was impractical for the commercial laboratory because of the duration and complexity of the test. In 1979 Bromhead developed a simpler device which forced the sample to shear against the upper platen (See figure 2.1e). This simplification avoided the mechanism necessary in the Bishop machine for controlling the size of the gap. The design also included a number of modifications to the layout of the machine that enabled tests to be conducted far more easily (Bromhead, 1979).

In shearing the sample against the upper platen, it must be assumed that the shearing resistance of the soil against the roughened porous platen is the same as the resistance of soil against soil. Providing the interface strength is equal or greater than the internal strength of the soil, this assumption would be correct. Tests forty years earlier by Hvorslev had suggested that this was an acceptable simplification. (Hvorslev, 1939). A series of tests using both the Bishop and Bromhead apparatus on the same soils gave identical results confirming the validity of Bromhead's simplification (Bromhead & Curtis 1983).

The research reported here is based on results using a Bromhead type apparatus.

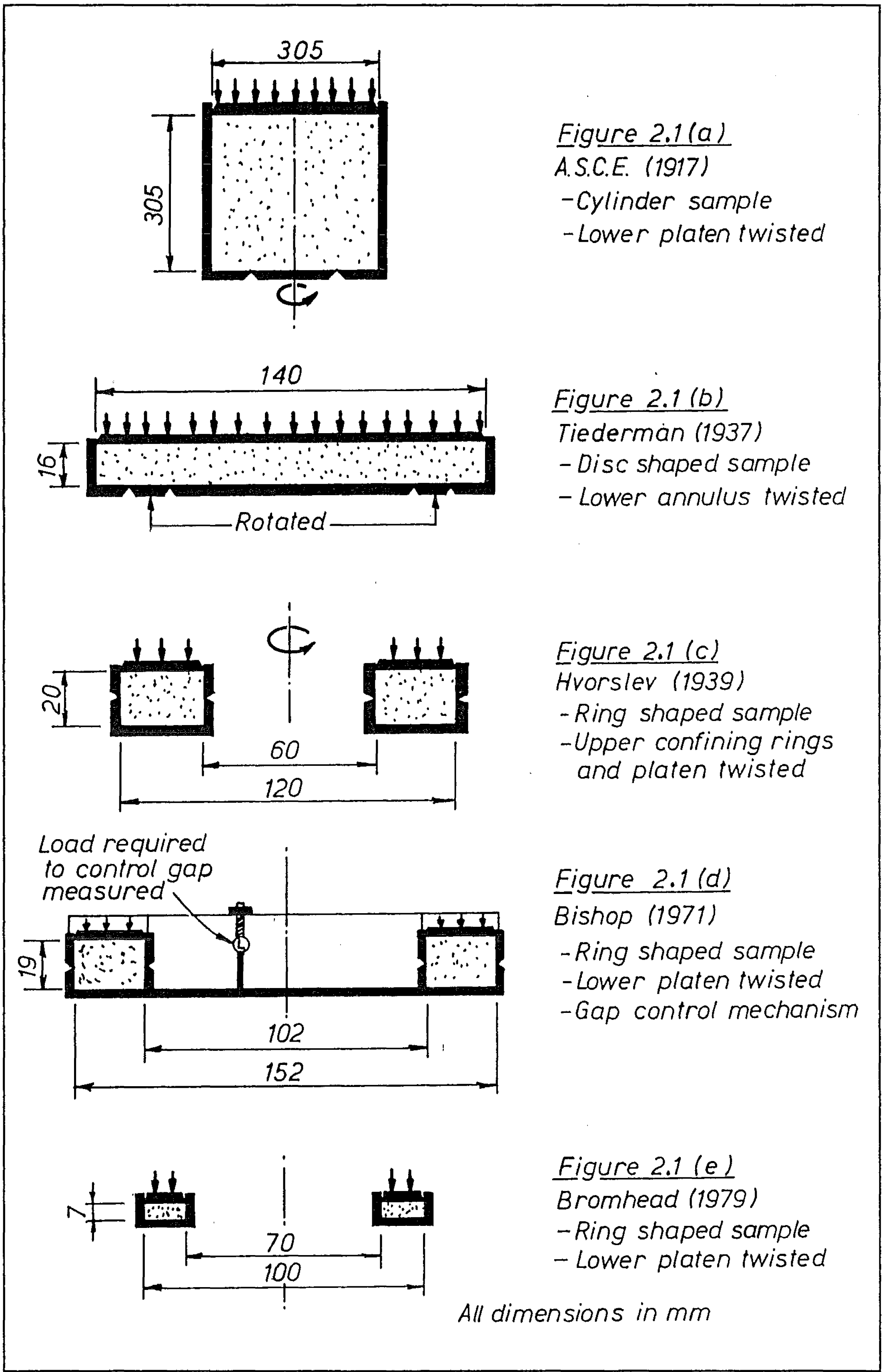



Figure 2.1 Development of Ring Shear Apparatus

## 2.2 DESIGN MODIFICATIONS

A Bromhead ring shear apparatus was imported from the English manufacturer, Wykeham Farrance. The layout of the loading cell is shown in Figure 2.2. The annular shaped sample is confined within an aluminium base and two loading platens. The upper loading platen is mounted to the head and torque arm and is horizontally restrained by a bush and central shaft. The upper platen, top and torque arm are free to move in the vertical direction. The lower platen fits within the base which is mounted to the machine chassis with a bearing assembly.

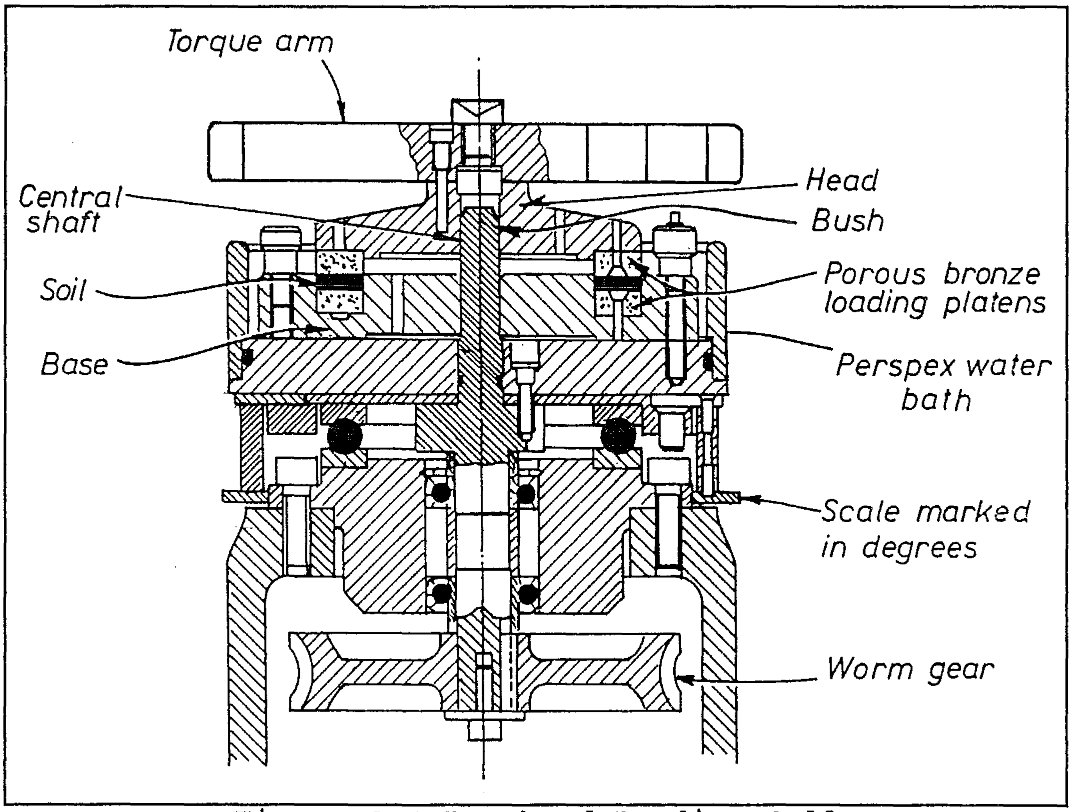



Figure 2.2 Bromhead Loading Cell

The soil is subjected to a normal load via the upper platen. The loading system consists of a hanger rod onto which weights are placed as illustrated in Figure 2.3. The 10:1 lever arm transfers the gravity force to the loading yoke and head. The shear load is initiated by rotation of the base and lower platen by the worm gear. This is connected by a chain and reduction gear box to an electric motor and provides for 25 different shearing rates from 60 to 0.024 deg/min. When the base is rotated clockwise the torque arm attached to the head comes into contact with the two proving rings or load cells. The shear stress can be determined from the summation of the measured force on each of the proving rings or load cells and the moment arm distance. Vertical movements in the head are also measured to determine whether the sample is dilating or consolidating.

Modifications to this machine were necessary for two reasons. Firstly, a number of design faults and manufacturer errors were encountered in using the apparatus for standard testing. This meant that significant alterations had to be made to the machine to enable simple measurements of residual shear strength. More complex alterations were also necessary to enable more accurate measurements of the effects of different shearing speeds.

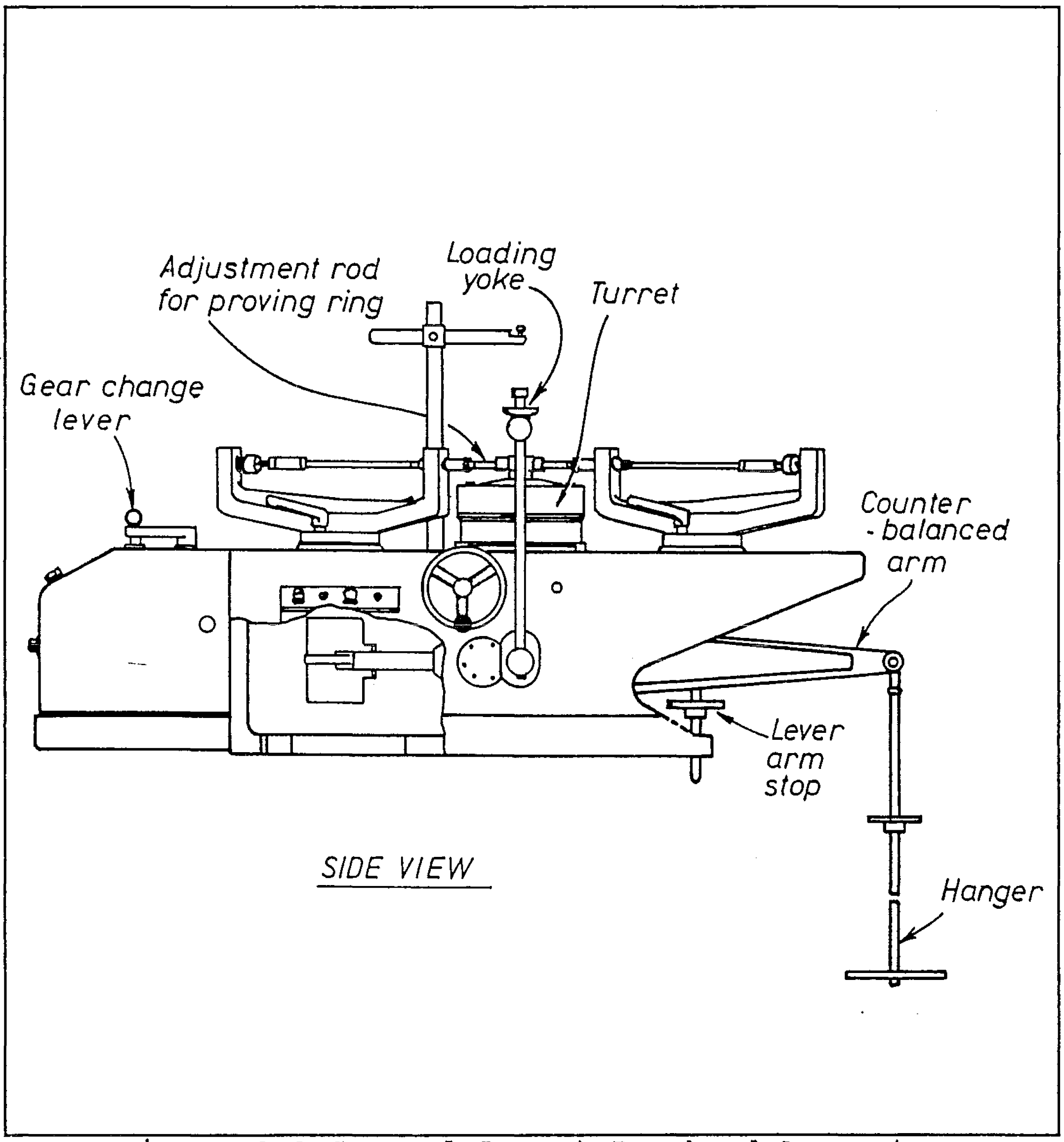



Figure 2.3 General Layout Bromhead Apparatus

### 2.2.1 Manufacturer Errors

The first manufacturer error detected was the incorrect centring of the base by 0.2 mm. This was sufficient to cause direct contact between the upper loading platen and the base during testing. This was corrected by rebuilding the loading base with a carefully centred bush.

A more difficult error to detect was the lack of drainage in the "porous" brass loading platens, which were found to be completely impermeable. This was suspected when the test results matched those of an undrained test and was confirmed by permeability tests on the platens.

Finding a suitable porous medium to replace the original brass platens proved difficult. A moulded mixture of coarse sand, fine sand and glue was tested extensively. The correct mixture of sand and glue was difficult to achieve. With too little glue, the medium had insufficient strength to withstand the stresses of testing. With too much glue, the platens rapidly became blocked and impermeable. This approach was abandoned in favour of certain modern porous plastic products.

Vyon, a semi-rigid micro-porous polymer material, was found to be a suitable product for the drainage surface. This material is supplied in sheet form and can easily be cut into the annular shape required for the apparatus. Although the Vyon surface has a texture akin to sand paper it was suspected that this might not be sufficiently rough and hence 1.5 mm teeth were machined into top and bottom drainage platens. Once the Vyon platens are made care must be taken to ensure they are not damaged. On tests where the sample was completely extruded, continued shearing resulted in the upper and lower Vyon surfaces rubbing and in some cases sufficient smoothing occurred so that the Vyon had to be replaced.

### 2.2.2 Gap Magnitude

A critical dimension in the Bromhead apparatus is the clearance between the base and upper loading platens. In contrast to the Bishop apparatus which allows the gap size to be adjusted, in the Bromhead apparatus this is fixed. Measurements of the residual strength may be significantly overestimated by the gap being too small and soil particles jamming. Conversely, too large a gap will allow the soil sample to extrude too quickly. The complications resulting from this extrusion process are described in detail in Chapter 3.

The ideal gap size will depend on the maximum particle size of the soil being tested and the loads being measured. A wide range of gap sizes was experimented with and the optimum was found to be in the range 0.2 to 0.4 mm. It should be emphasized that these trials utilised one soil type (Temuka Clay) and this may not be the ideal gap size for all soils.

It was also found advantageous to shape the upper platen so as to minimize friction without allowing extra soil to extrude. A fine lip was added to the lower edge of the upper platen. An approximate analysis of the friction developed in this gap is given later in this chapter (See Section 2.5.3 and Figure 2.14).

### 2.2.3 Load Measurement Imbalance

With the modifications described above it was still not possible to obtain a steady minimum shearing resistance from the Bromhead apparatus. When used to test a silty clay, the load oscillated from one load cell to the other as the sample was rotated causing a significant variation in the measured shear stress. Even when the load cells were re-balanced, the problem would recur after a small displacement. Subsequently this problem has been noted by other users of the apparatus. Researchers from Loughborough University of Technology note :

> A problem was encountered when shearing commenced. Readings on the two proving rings measuring torque were initially equal, but as rotation proceeded they gradually became more unbalanced and eventually one reading fell to zero.
>
> Anayi, Boyce and Rogers (1989b) p. 171.

This load imbalance in the head will introduce friction on the central locating shaft and undesirable normal stress variations in the soil. To analyse these effects consider the head as a free body as illustrated in Figure 2.4.

In the vertical direction, the head is subjected to a point load from the loading yoke, P, which is resisted by an assumed uniform normal stress in the soil. The shear

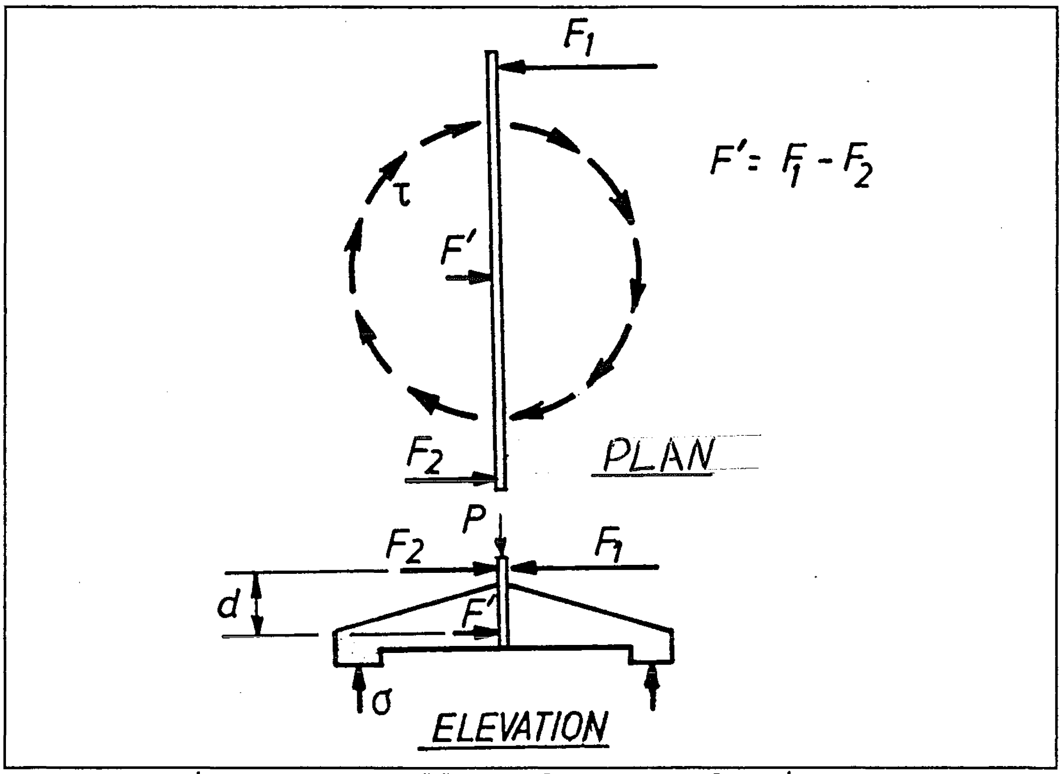


Figure 2.4 Cell Head Free Body Diagram

stresses generated in the soil by the rotation of the base will be resisted by the two loading rods resting against the torque arm. If these loads, $F_1$ and $F_2$, are equal there will be no reaction on the shaft and the system will function correctly.

If, however, $F_1$ does not equal $F_2$, the shaft will have to provide a reaction F′. This force will occur on a plane at the level of the bush, which is below the level of the torque arm and loading rods by a distance d. This suggests that not only does the force imbalance cause a force on the locating shaft but it also introduces a moment, M, to the head.

$$M = (F_1 - F_2)\, d \qquad \text{Eqn 2.1}$$

This moment is resisted either by a variation in the normal stress distribution about the circumference of the sample or an equivalent moment acting on the central locating shaft. Depending on the tolerances in the bush and the stress/strain characteristics of the soil, this moment will be shared between these two mechanisms.

Assuming large tolerances in the bush, all of this moment will be resisted by the soil. The likely variation in the normal stress can be estimated by assuming a linear stress/-strain relation in the soil. This is a reasonable approximation providing the stress variation is small. This assumption implies that the stress will vary proportionally to the distance from the centre axis.

$$\delta\sigma = c\ x \qquad \text{Eqn 2.2}$$

where $\delta\sigma$ = variation in normal stress from mean

$c$ = constant

$x$ = distance from axis of rotation

This function can be integrated over the annular area of the sample as illustrated in Figure 2.5. The maximum variation in stress from the average normal stress caused by this moment can then be determined.

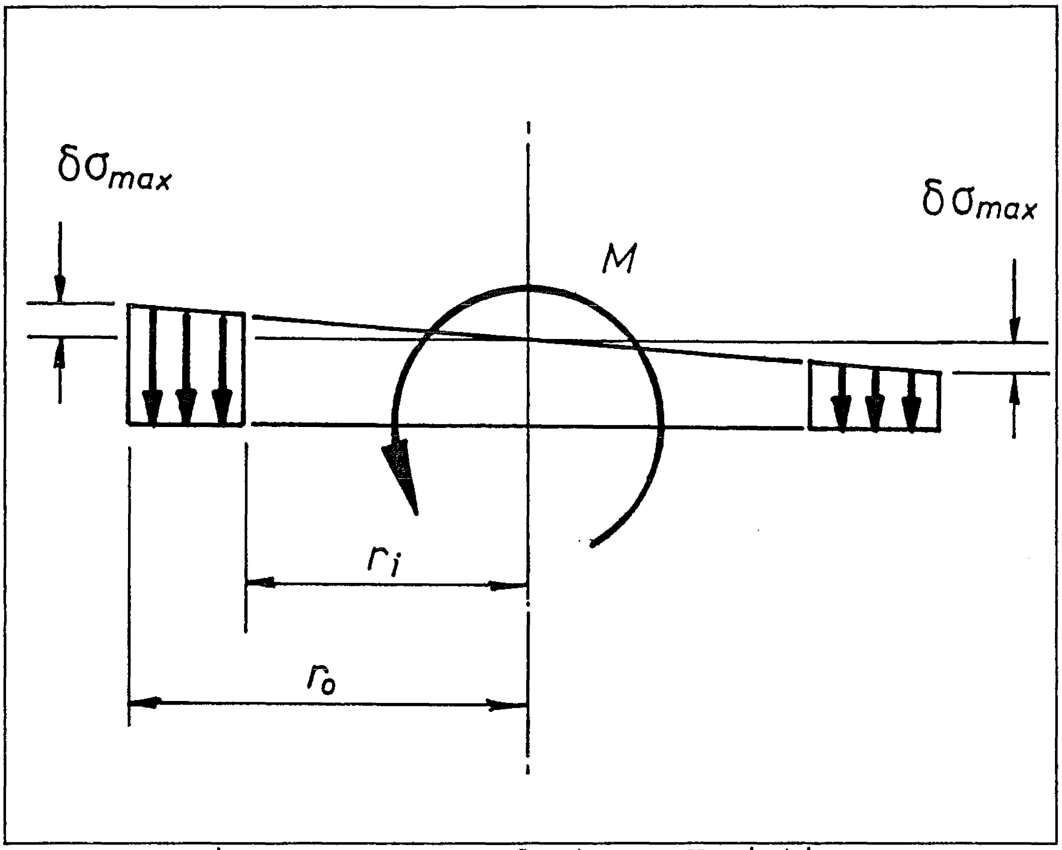



Figure 2.5 Normal Stress Variations

$$\delta\sigma_{max} = \frac{4\ M}{\pi\ (r_o^3 - r_i^4/r_o)} \qquad \text{Eqn 2.3}$$

where $\delta\sigma_{max}$ = maximum variation in normal stress

$r_o$ = outer radius of upper platen

$r_i$ = inner radius of upper platen

The maximum variation in the normal stress can then be determined by substituting Equation 2.1 into Equation 2.3.

$$\delta\sigma_{max} = \frac{4\ (F_1 - F_2)\ d}{\pi\ (r_o^3 - r_i^4/r_o)} \qquad \text{Eqn 2.4}$$

To illustrate the magnitude of this problem, consider the testing of a soil with zero residual cohesion and a residual friction angle of 25°. If tested at a normal stress of 100 kPa, each load cell would ideally be loaded to 52.8 N. In practice this will not occur and the load will be shared unevenly. For example, one load cell may take 70 N and the other 35.6 N. This would cause a moment, as described in Equation 2.1, of 0.7 Nm. This moment would distort the originally uniform normal stress of 100 kPa to a stress pattern that would vary from 90.8 to 109.2 kPa.

The variation in normal stress will cause equivalent variations in the shear stress and further imbalances in the head. Pore pressure complications may also result from the periodic normal stress variations which will occur with each rotation. The bulk of the soil sample rotates with the base such that any element of soil will be subject to continuous changes in the normal stress. Depending on the rate of rotation and the consolidation characteristics of the soil, there may be insufficient time for pore pressures to dissipate and a complex pore pressure regime may develop. Clearly any variations in the normal stress will cause undesirable distortions in the measurement of residual shear strength.

If the imbalanced moment is partially restrained by the central locating shaft, the normal stress variation will be less. However, the effect of this moment on the shaft is also undesirable as it will increase the friction between the bush and the shaft.

This problem can be eliminated by redesigning the head so that the torque arm, locating bush, and the soil shearing zone are located on the same plane. Mathematically this equates to making the distance, d, equal to zero. This results in a zero moment in Equation 2.1 regardless of any load difference between $F_1$ and $F_2$.

This plan allows for the possibility of eliminating one loading arm altogether, a move which would make the system statically determinate and simplify the apparatus considerably. It would, however, introduce a large horizontal load on the central shaft. This would be equal, but of opposite direction, to the load on the remaining loading arm. Two problems may result. Firstly, since friction is proportional to the normal load, it may introduce unacceptable levels of friction into the system. This problem will be addressed in the following section.

The second possible problem is that the load on the shaft may cause sufficient deflection in the shaft for contact to occur between the upper platen and the base. Estimations of the possible deflections of the shaft at the centreline of the bush relative to the base can be made using elastic analysis. These indicate that the deflections will be proportional to the shear load with a constant of proportionality of 0.00053 mm/N. Testing a soil with a residual angle of internal friction of 25° at 400 kPa using only one loading arm will cause the head to deflect 0.22 mm relative to the base. This movement does not present a problem, providing the magnitude of the tolerances between the upper platen and the base is larger.

The concept of removing one loading arm altogether has several advantages. It eliminates the statically indeterminate nature of the load measurement system and thus avoids the problem of the load oscillating from one load cell to the other. It also simplifies both the set-up and the shearing stages of testing. It may, however, introduce intolerable errors caused by friction in the bush. An analysis to determine the magnitude of these errors is necessary before this alteration can be considered.

### 2.2.4 Friction

Bromhead (1979) correctly determined that friction between the central shaft and head bush would have two effects on the measurement of residual strength. Firstly, friction in the vertical direction on the bush will reduce the normal load on the sample, thus reducing the measured shear strength. The second effect is that friction will occur about the surface of the bush causing a small moment. This moment will cause the measured strength of the soil to be greater than the actual strength. Qualitatively, Bromhead concluded that the two errors tend to cancel each other.

With the proposed design changes to the machine and the requirement for very accurate measurements in this research, a more quantitative analysis of the effects of friction is required.

The frictional force between the shaft and the bush will be proportional to the horizontal load.

$$F_f = (F_1 - F_2)\ \mu_b \qquad \text{Eqn 2.5}$$

where $F_f$ = frictional force between shaft & bush

$F_1, F_2$ = loading arm forces as per Figure 2.4

$\mu_b$ = coefficient of friction between shaft & bush

As the head moves down, this frictional force will act in the vertical direction and will result in a reduction in the normal load on the sample.

$$\delta\ \sigma_f = -\frac{F_f}{\pi\ (r_o^2 - r_i^2)} \qquad \text{Eqn 2.6}$$

where $\delta\ \sigma_f$ = change in normal stress due to friction between shaft and bush.

$r_o, r_i$ = outside and inside radius of sample as per Eqn 2.3.

The reduction in normal stress will result in a reduction in the measured shear strength of the soil.

$$\delta\ \tau_{f1} = -\ \mathrm{Tan}\ \emptyset_r\ \delta\ \sigma_f \qquad \text{Eqn 2.7}$$

where $\delta\ \tau_{f1}$ = change in measured shear strength due to shaft friction.

$\emptyset_r$ = residual friction angle

As the base rotates, a frictional moment will develop on the surface of the shaft. The magnitude of this moment will equate to the frictional force acting on the shaft multiplied by the moment arm (in this case the shaft radius).

$$M_f = F_f\ r_s \qquad \text{Eqn 2.8}$$

where $M_f$ = Moment caused by friction between shaft and bush.

$r_s$ = radius of shaft

This frictional moment will add to the load being measured by the load cell or proving ring and will result in the shear strength of the soil being overestimated.

$$\delta\ \tau_{f2} = \frac{2\ M_f}{\pi\ (r_o + r_i)\ (r_o^2 - r_i^2)} \qquad \text{Eqn 2.9}$$

where $\delta\ \tau_{f2}$ = change in measured shear strength due to friction moment.

Adding the two friction effects from equation 2.7 and 2.9 together yields the net effect of friction on the shaft.

$$\delta\ \tau_f = \frac{(F_1 - F_2)\mu_b}{\pi(r_o^2 - r_i^2)} \left[ \frac{2r_s}{r_o + r_i} - \text{Tan}\emptyset_r \right] \qquad \text{Eqn 2.10}$$

where $\delta\ \tau_f$ = net effect of friction on measured shear strength.

Bromhead's assumption that the two effects cancel is only true when the term inside the brackets equals zero. Substituting the dimensions of the apparatus, this equates to a soil with a residual shear strength of 8°. Soils of greater strength than this will result in the strength being under estimated. Conversely, for soils of lesser strength than 8°, the strength will be overestimated.

To determine the likely magnitude of these errors, the friction coefficient between the shaft and the bush is required. The coefficient, $\mu_b$, was determined to be approximately 0.38 by measuring the moment with no soil in the apparatus but with a horizontal load. In section 2.2.3, a soil with a residual friction angle of 25° and zero cohesion was used as an example. The analysis looked at the effect on the normal shear stress distribution of one load cell measuring 70 N and the other 35.6 N. Using this same example, and Equation 2.10, the shear stress would be underestimated by 1.06 kPa. This amounts to an error of 2%.

If we consider the same example, but remove one load cell as proposed in Section 2.2.3, the error would increase to 3.25 kPa or 7%. These errors are becoming unacceptably large and so further design alterations were considered to correct them.

The use of ball bearings in this critical area of the machine was a logical way of minimising friction. As both vertical and rotational movement are required, a combined system of a linear and needle roller bearings was used. The coefficient of friction for the bearing system is 0.0025 (SKF Bearing Catalogue 1975).

In the example described, this would reduce the error in the measurement of the shear strength to just 0.02 kPa or 0.05% with the modification of using just one loading arm. This magnitude of error was considered acceptable as it is significantly less than the accuracy of the load cells.

The bearing system did create some additional practical problems. Fine soil particles, if allowed into the bearing system, would rapidly damage the bearings and distort the readings. The use of bearing seals was not an acceptable solution as these would introduce excessive amounts of friction back into the system. To overcome this problem the base was redesigned to provide a catchment for the extruded soil. Furthermore, to optimise the performance of the bearings, an oil bath was introduced to the design of the base. This ensured that the critical linear and needle roller bearings were well lubricated. It also minimized the corrosion of the bearings which was a strong possibility in the moist environment. This and the other alterations are illustrated in Figure 2.6.

While the bearings enabled very accurate measurements of the residual strength, it was found impossible to keep a perfectly clean environment in the vicinity of the bearings. Several times during this research work the bearings had to be replaced. A design more suitable for work not requiring such high levels of accuracy would replace the roller bearing system with one of the many modern plastic bearings available. With most having a friction coefficient of about 0.08, they would provide measurements of the residual strength of acceptable accuracy. (1.5% error in the same example de-

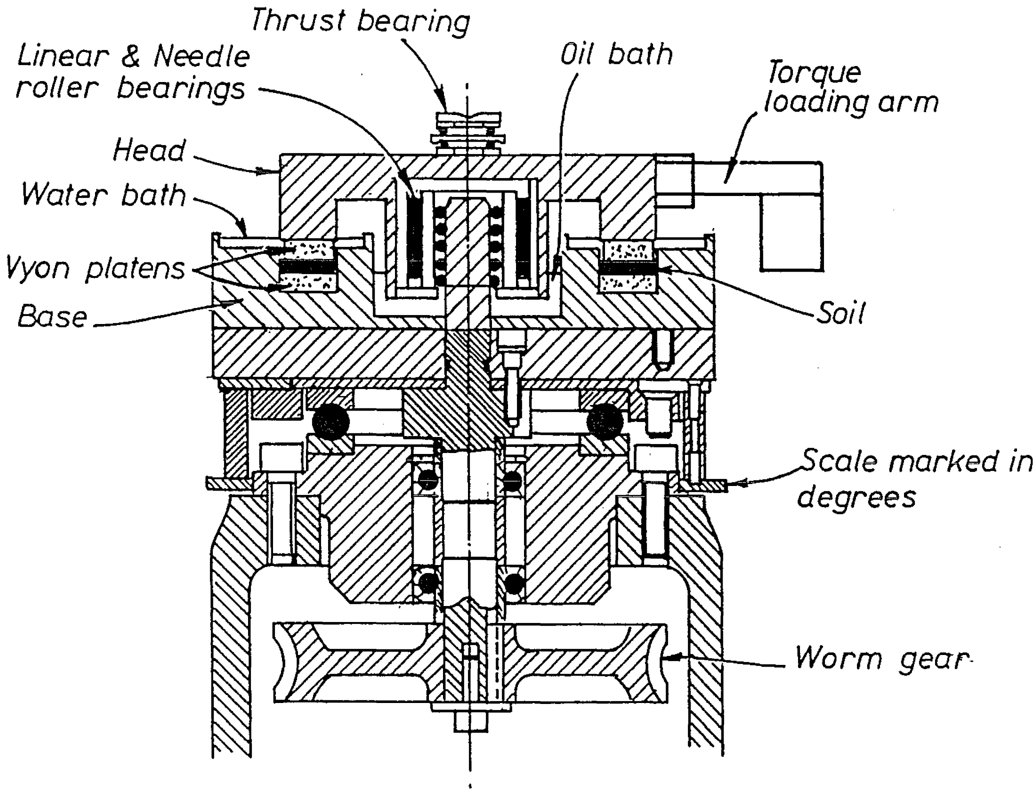



Figure 2.6 Modified Bromhead Loading Cell

scribed earlier). This avoids the inconvenience and cost of having to replace the bearings regularly but gives more accurate results than the original brass bush.

The contact point between the loading yoke pin and the head is a further source of friction and errors. While the movements at this interface are small, they will be sufficient to allow friction to develop and the shear stress measurement to be distorted. The magnitude of these errors can not be easily determined as the contact area is ill defined and the displacements are small. Estimations indicate they may be as high as 8%. To avoid the uncertainty that this brings to the results a small thrust bearing was introduced between the loading yoke pin and the head.

With the modifications described the apparatus was used to measure the residual strength of Temuka Clay (See Appendix I for full description of soil). The sample was over-consolidated to 300 kPa and then sheared at a normal stress of 100 kPa. At a shearing speed of 46 mm/min, the sample was sheared to a total displacement of 70 m. A complete description of the test procedure and the results are given in Appendix IV. Figure 2.7 summarizes these results and illustrates the success of the modifications described.

### 2.2.5 Drainage Measurements

Information regarding whether water enters or exits the sample during shearing would assist in understanding the behaviour of the soil in the ring shear apparatus. In the triaxial apparatus, the sample is completely sealed. This allows the soil to be tested under drained or undrained conditions and for water volume changes or pore pressures to be measured. The sample in the ring shear apparatus cannot be completely sealed as the gap about the perimeter of the head is necessary for the load measurement system. However,

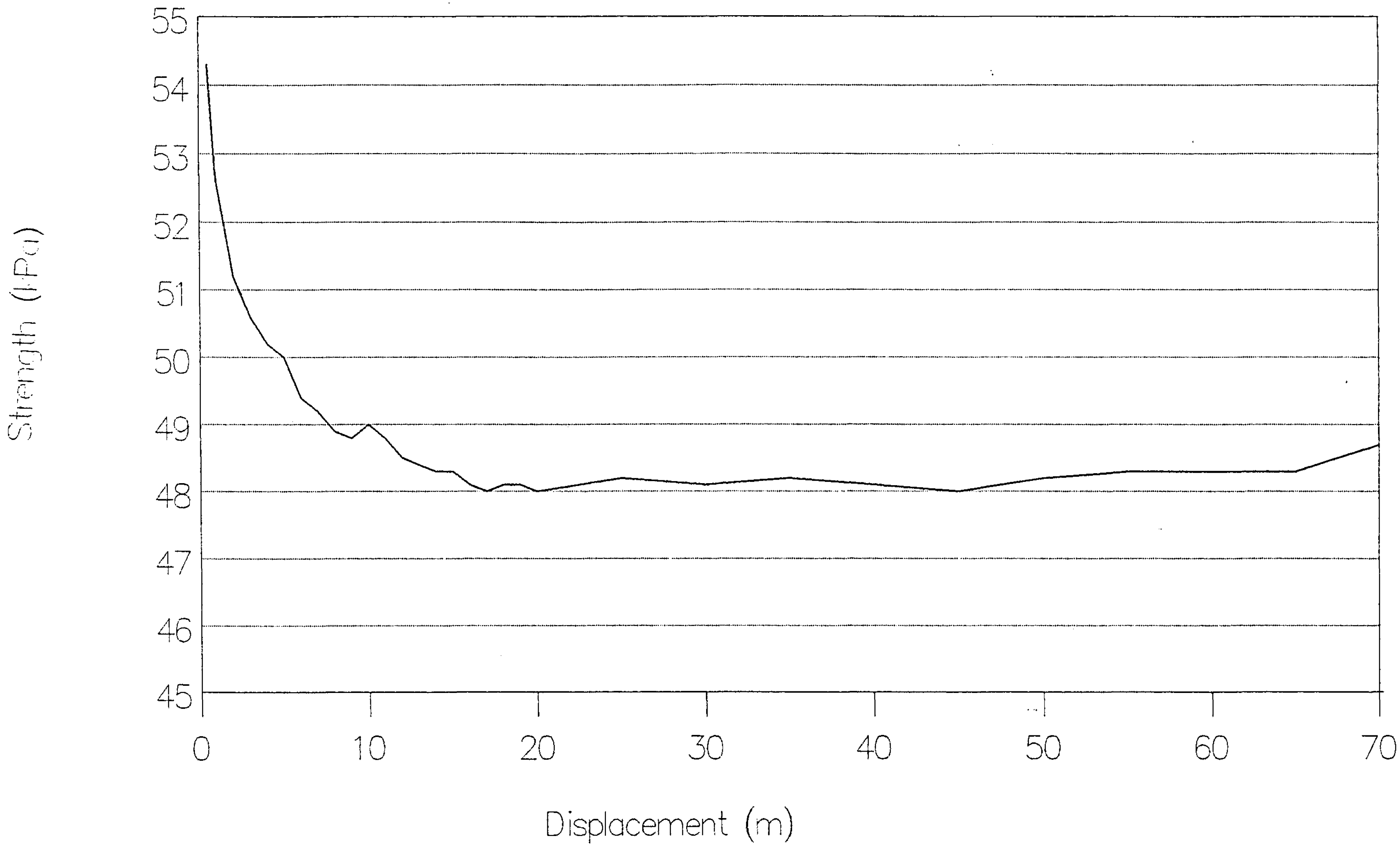



Figure 2.7  Ring Shear Test #1

approximations of water volume changes are possible with some modifications.

Three drainage paths are open to the soil sample in the unmodified machine. Water may exit or enter the sample through the lower platen, through the gap between the platens or through the upper platen. The connection of a burette to the lower platen is fraught with complications because the base rotates. To avoid these difficulties, the drainage paths to the lower platen may be blocked. With the geometry of the sample being that it is considerably wider than its thickness, and with the shearing layer being adjacent to the upper platen, the bulk of the water exiting or entering the sample will do so through the upper platen. This will be particularly so once shearing has commenced and the gap has become filled with extruding soil. By sealing the upper platen to the head and plumbing this to a manifold and burette it became possible to measure the movement of water to or from the sample (See Figure 2.8).

As the water level in the burette changes, its level needs to be adjusted so as not to introduce a pressure head to the sample or to cause flows that will distort the water volume measurements. Mounting the burette on a height adjustable frame allows the burette to be manually adjusted to ensure that the level is the same as in the water bath. The burette enables the measurement of sample water demand. This data

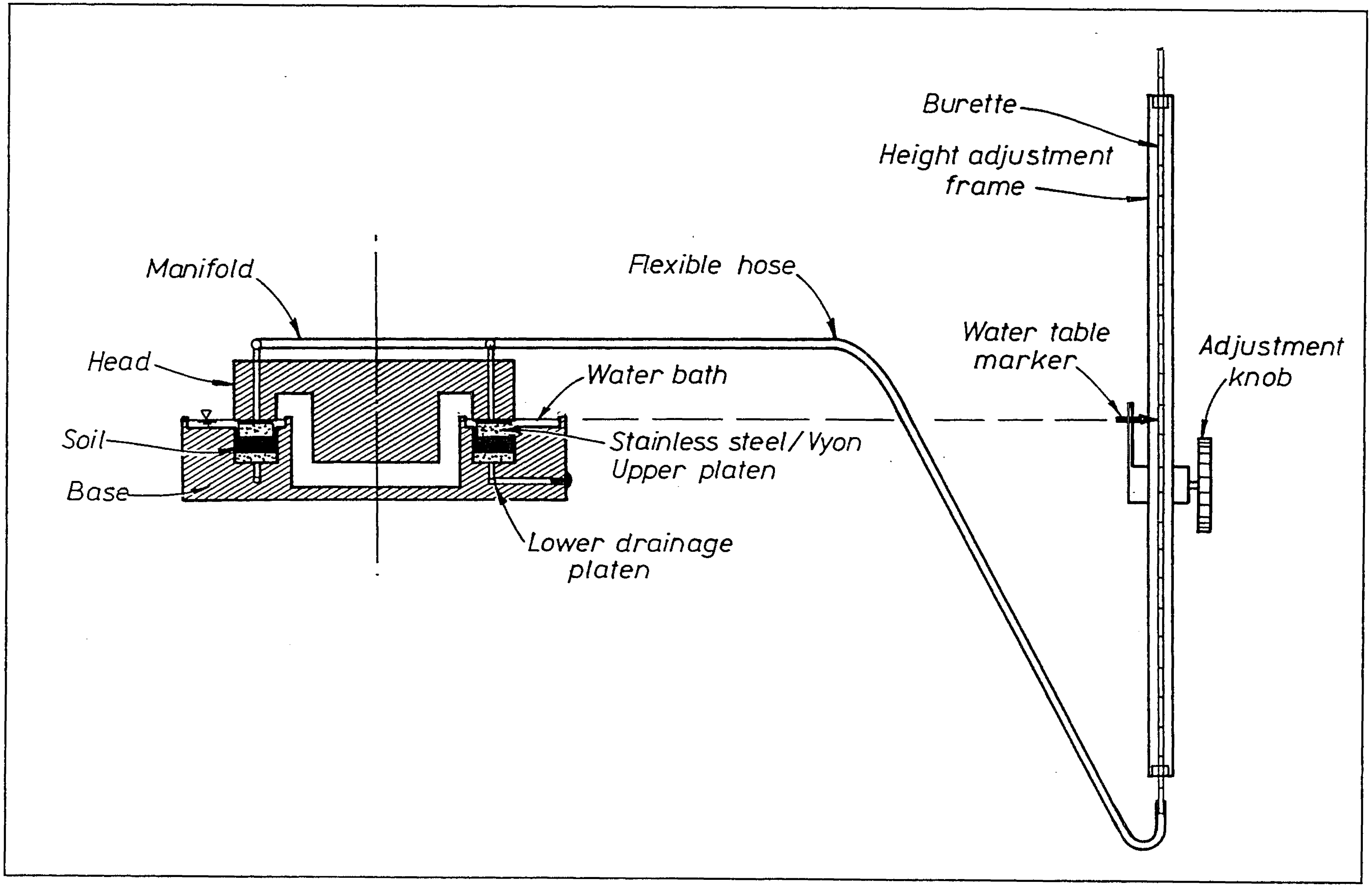



Figure 2.8 Drainage Measurement System

will be critical in understanding the soil behaviour observed in rate effect studies by other researchers.

### 2.2.6 Range of Shearing Rates

In order to study rate effects on the residual shear strength, modifications were needed to enable the machine to test at both faster and slower rates. Two additional driving mechanisms were required to enable the machine to operate at these different rates: a secondary drive to extend the fast range, and a tertiary drive to extend the slow range.

The maximum shearing rate of the original machine was 46 mm/min, yet actual landslides are known to have moved at rates greatly in excess of this. The East Abbotsford landslide, for example, had a maximum velocity of 2 - 3 m/min.

A secondary motor and drive system were introduced to the apparatus as the existing motor had insufficient power to drive the system at the necessary rates. A variable speed motor was connected directly to the worm drive as detailed in Figure 2.9. This enabled the sample to be sheared at rates up to 4 m/min.

Modifying the machine to allow significantly slower shearing rates was more complicated than extending the range into the faster rates. It was first tried by introducing secondary

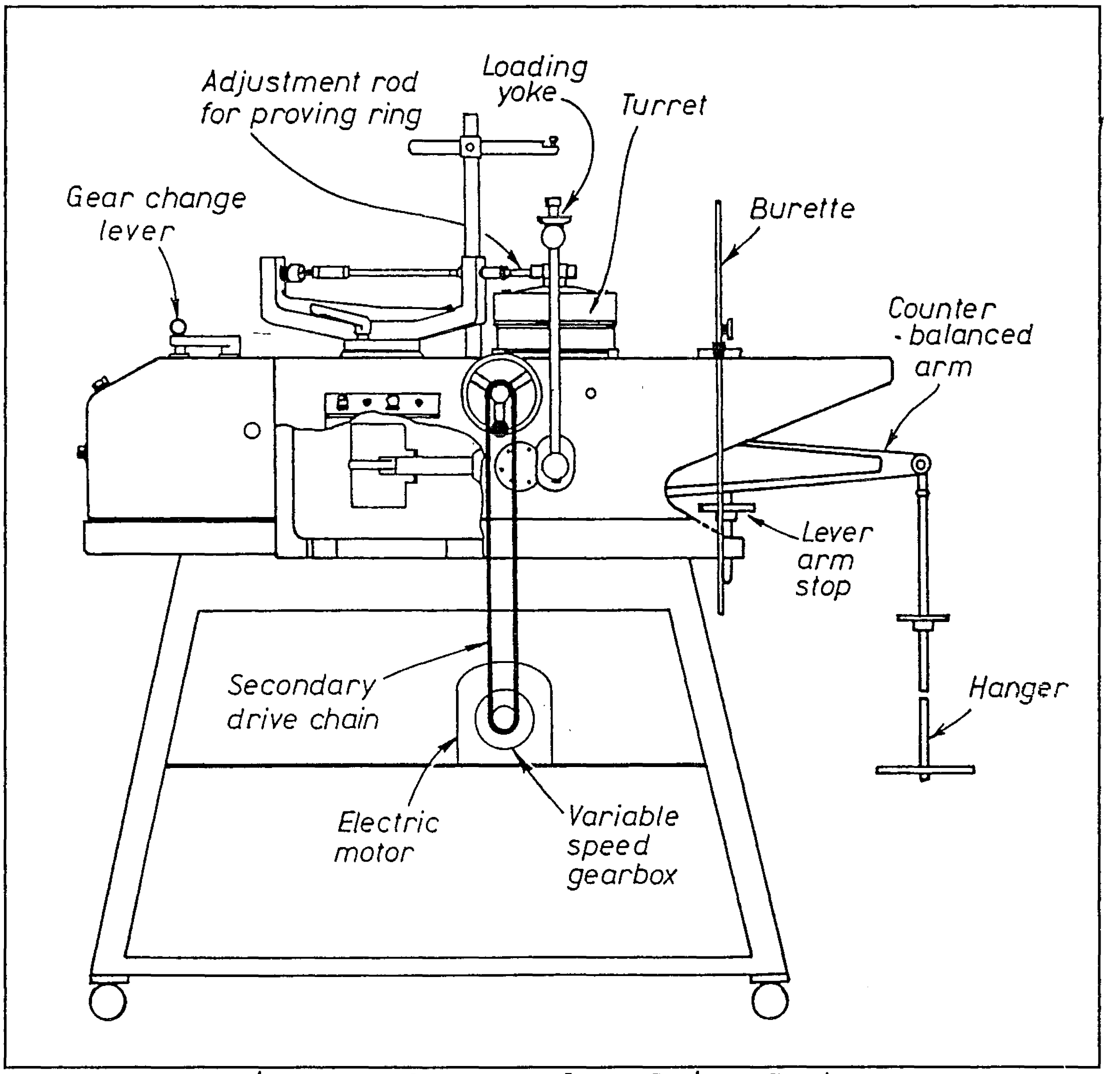



Figure 2.9 Secondary Drive System

gears into the existing drive system. This was found to be quite unsatisfactory as the worm drive was incapable of producing very slow continuous movements. Instead, a separate tertiary drive was introduced that bypassed the worm drive and rotated the head rather than the base. This is illustrated in Figure 2.10.

A separate mounting plate was attached to the back of the machine onto which the tertiary motor, control panel and reduction gearbox was bolted. The stationary load cell mount

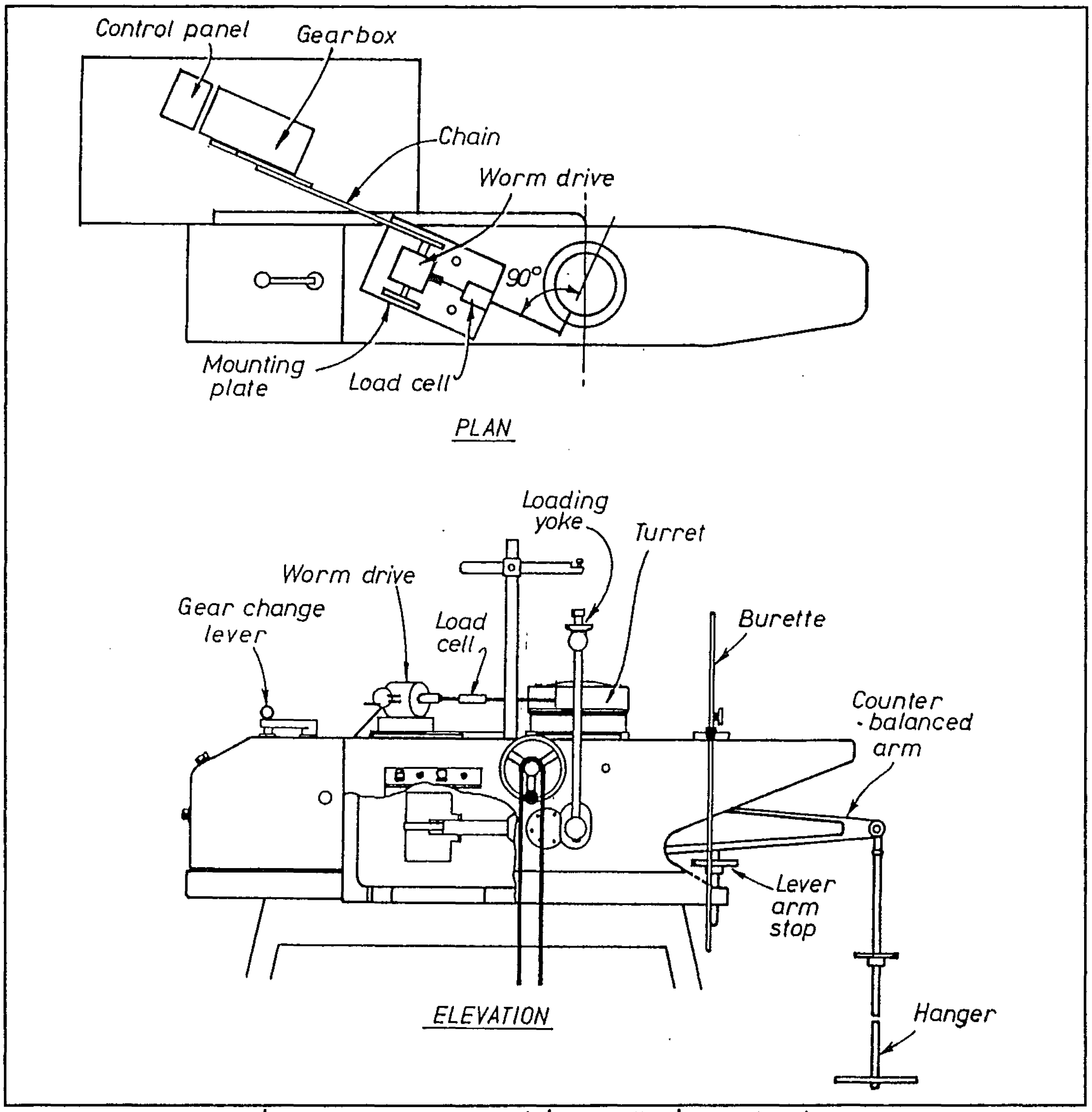



Figure 2.10 Tertiary Drive System

on the original machine was replaced with a worm drive which could be screwed either manually or by the chain drive connected to the gearbox. Movement in the worm drive forces the load cell and loading arm to rotate the head. The shear movement that this generates in the soil is less than the movement in the worm drive and is determined by measurement of the radii.

The load measurement system depends on a right angle between the torque arm and the load measurement rod. As the head is rotated using the very slow drive this angle is distorted and errors are introduced into measurements of the shear stress. These errors are quantified in Section 2.5.4 where it is shown that shear displacements using the tertiary drive are possible up to 2.5 mm either way without causing significant errors in the shear stress measurement.

The limitation of only being able to shear the sample for small displacements using the tertiary drive is particularly restrictive when several stages of slow shearing are required on one sample. This problem can be overcome by reversing the tertiary drive and returning the head to its former position. Normally if this is done, the shear load will be removed, causing further complications. However, if the head is reversed and the base rotated simultaneously, the shear load can be maintained. The simplest way to accommodate this process into the testing schedule is to simultaneously reverse the tertiary drive at the same time the primary or secondary drive is operating. The true rate of shear occurring within the shear zone of the soil can be determined by the difference between the two rates. Typically, the tertiary drive reversal will be at such a slow rate relative to the movement in the base that the difference can be ignored.

The process of forwarding and reversing the tertiary drive may cause distortions in the study of the rate effects. While operating in the forward direction, the thrust bearing between the loading pin and the head will provide a small frictional moment that will add to the measured shear stress. In the reverse direction, the frictional moment will reduce the measured shear stress. As it would be easy to misinterpret these effects the thrust bearing system was modified as shown in Figure 2.11.

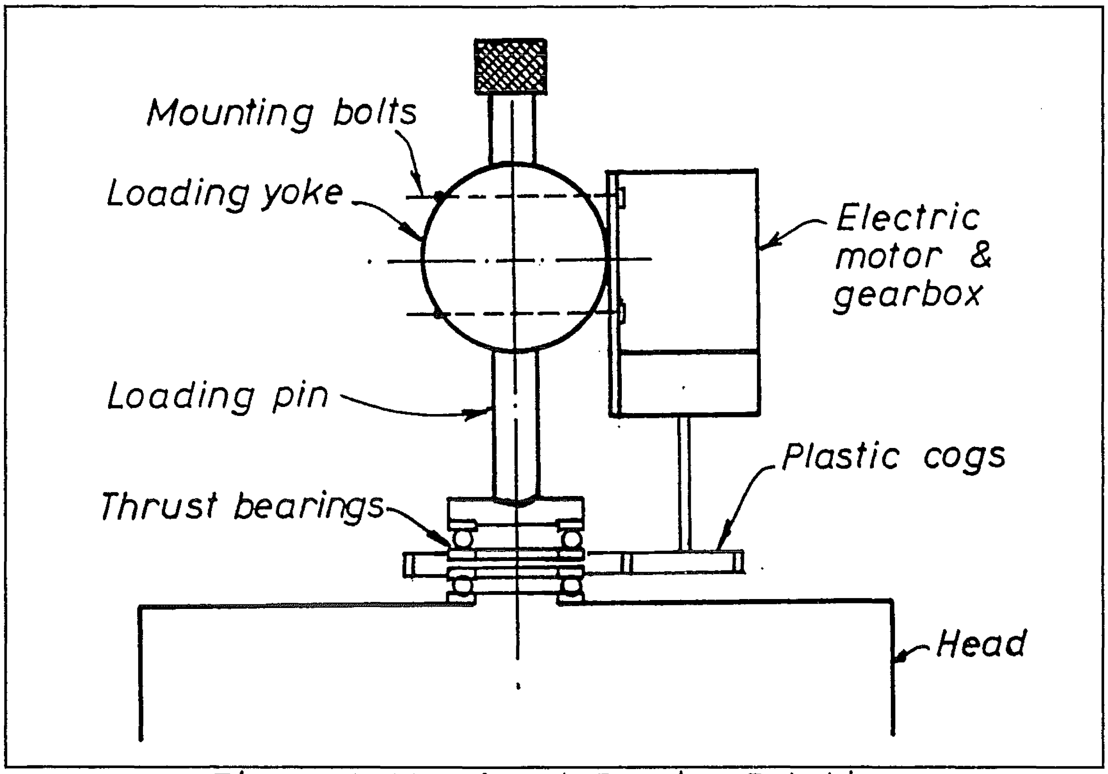
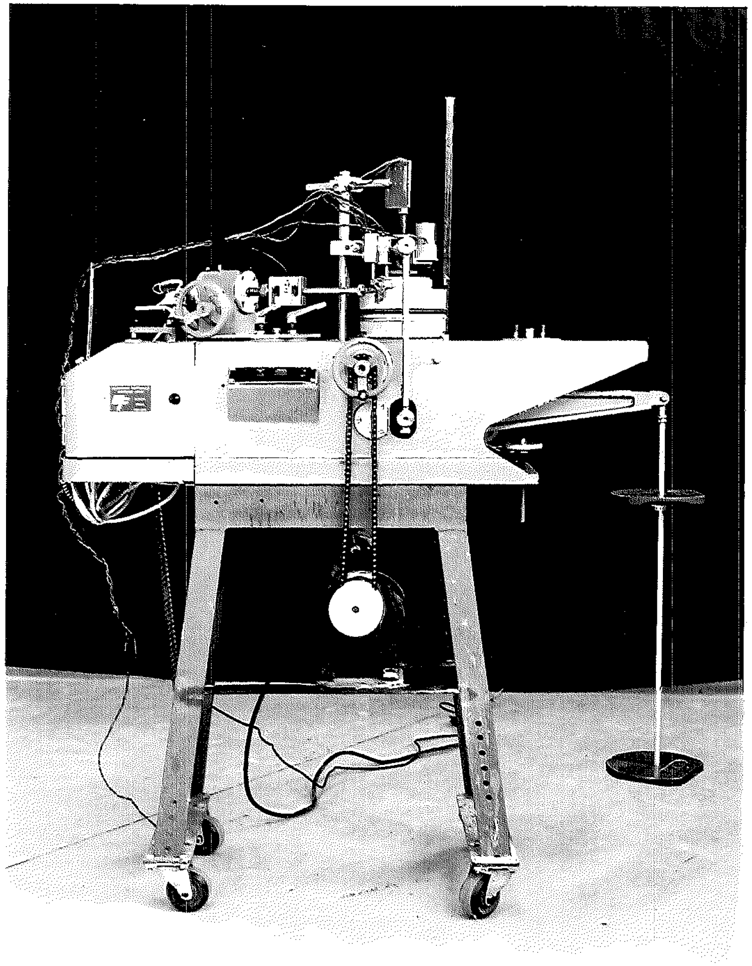



Figure 2.11 Thrust Bearing Rotation

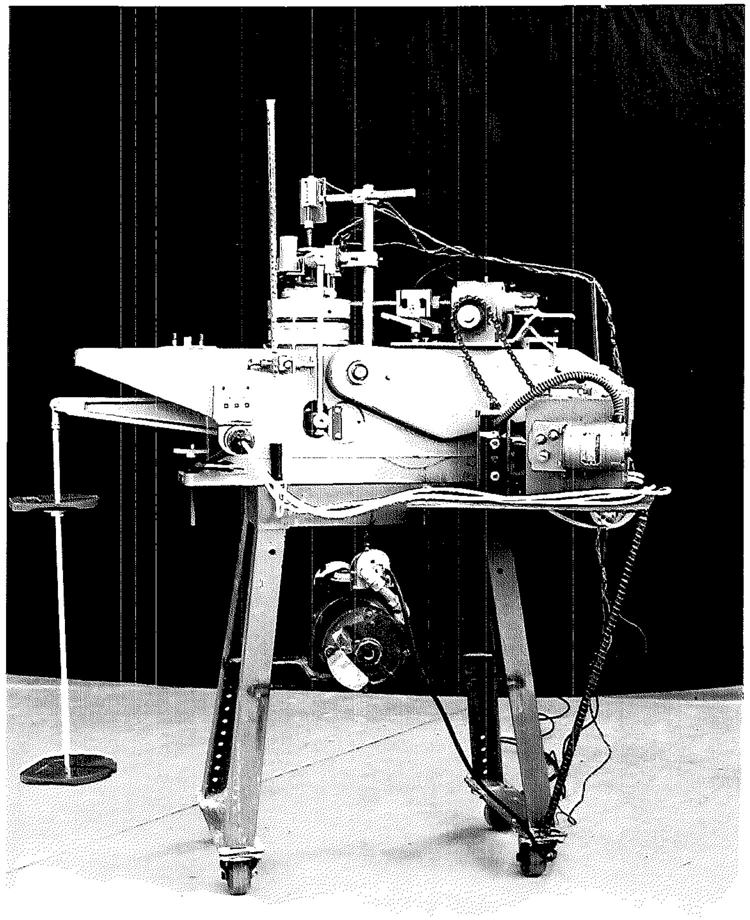
A small electric motor mounted on the loading yoke and connected to the middle of a double thrust bearing enables the bearing to be rotated continuously in one direction at a relatively fast rate. This ensures that any frictional moment caused by friction in the thrust bearing is always in one direction regardless of which of the three shearing drives is engaged.

With these modifications the ring shear apparatus could be used to test soils at shearing rates over a range of seven orders of magnitude. The slowest rate that could be generated was 0.4 mm/day while rates as high as 4 m/min were also possible. The modifications also removed many sources of error that could have distorted results in this study.

The two photographs on the following page show the modified Bromhead ring shear apparatus as used in this research project.



Photograph 2.1 Front View of Apparatus



Photograph 2.2 Rear View of Apparatus

## 2.3 INSTRUMENTATION

The instrumentation required for the modified ring shear apparatus was a load cell, four displacement potentiometers and a burette.

The load cell was used to measure the force in the loading rod. Knowing this load enables the torque and hence the shear stress to be determined. A shear type load cell manufactured by Precision Transducers Ltd was used (Precision ST 50 kg, Serial No. 0273). The shear type cell was chosen as it is the most suitable for applications where the load cell lies in the horizontal plane. The cell uses strain gauges to measure the shear strain in the "S" part of the cell. It is precision grade and has a combined error of 0.025%. Its maximum capacity is 50 kg. During testing, the load cell was excited with equal positive and negative voltage so as to maximise the accuracy of the data acquisition system (See Figure 2.12). The magnitude of the excitation voltage was varied for each of the tests depending on the normal stress so as to use the maximum range of the recording system. The load cell was calibrated in a vertical position using weights and then re-zeroed when mounted.

The first displacement potentiometer was mounted above the loading yoke assembly and rested on the adjustable loading pin. This enabled vertical movements in the head, either from sample extrusion or consolidation, to be determined.

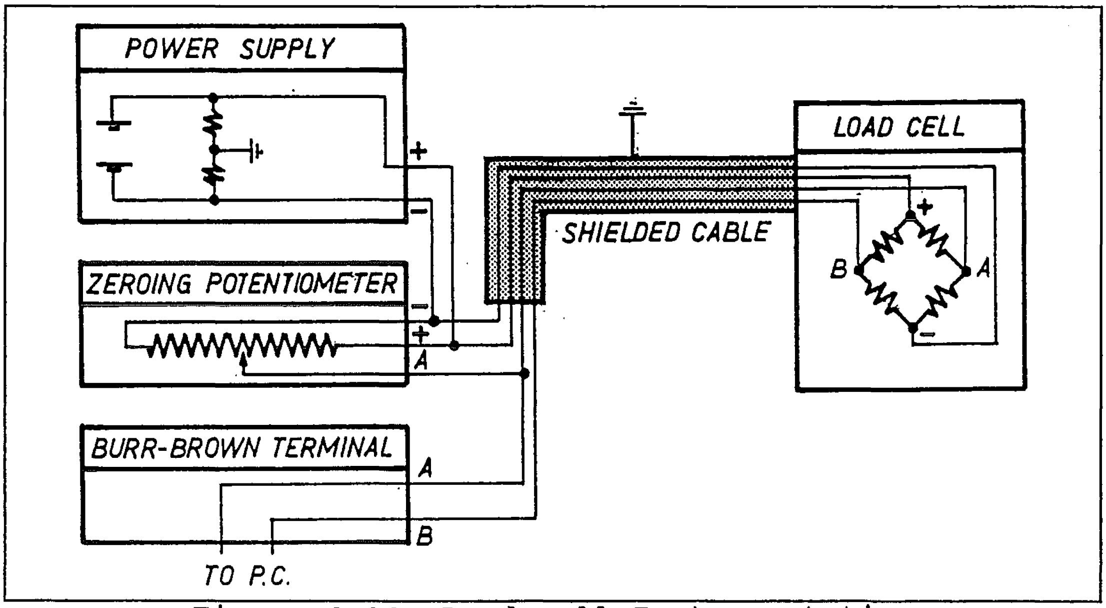



Figure 2.12 Load Cell Instrumentation

Two further displacement potentiometers were placed vertically on the head, ninety degrees apart, to provide data on any tilt of the head. The forth potentiometer was mounted behind the worm drive of the tertiary drive mechanism and provided an accurate measurement of the displacements while shearing with the tertiary drive system. The potentiometers had a maximum displacement of 15 mm and a linearity error of 0.02%. The potentiometers were excited with a separate voltage supply from the load cell to avoid any cross interference. The maximum excitation voltage was used so as to maximise the accuracy of measurements. The potentiometers were calibrated using 5 mm thick pieces of aluminium (See Figure 2.13).

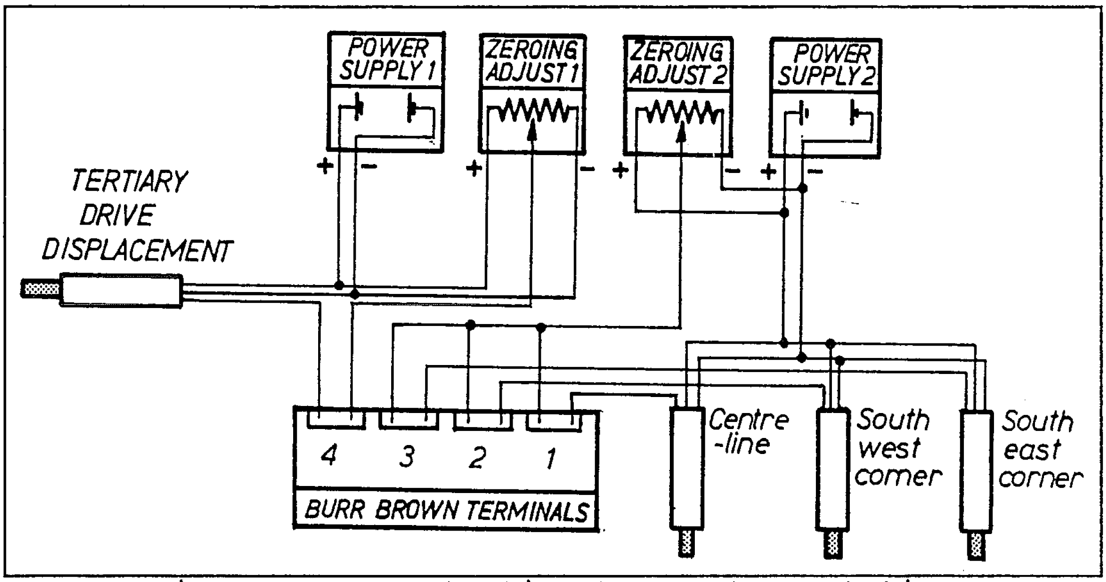



Figure 2.13 Potentiometer Instrumentation

The burette was used to measure drainage to and from the upper platen. Volumes could be measured to an accuracy of 0.1 millilitres.

## 2.4 DATA RECORDING SYSTEM

Data was recorded using an I.B.M. compatible personal computer with a Burr-Brown data acquisition board. The outputs from the load cell and potentiometers were connected to a terminal panel. To minimise the effects of interference, the terminal panel was earthed, the load cell cable was shielded and the potentiometer wires were woven together. The terminal panel was connected to the Burr Brown piggy-back board inside the P.C. with a multi-plied shielded cable.

The Burr-Brown board uses a 12-bit processor dividing the 10 volt maximum range into 4096 parts. The board includes an amplifier which can be programmed for gains of 1, 10, 100 or 1000. The largest gain of 1000 was used for the load cell and a gain of 10 was used on each of the four potentiometers.

A programme to allow flexible recording of the data was written. This enabled data to be collected at a rate up to 100 times per second and it could be automatically graphed on the screen, recorded on disk or printed on hard copy. The data was stored on floppy disks in integer form to enable fast rates of data collection. A further programme was used to process the data. This allowed the average and standard deviation to be determined over intervals and the removal of any wild points.

To ensure that temperature effects in the load cell or computer hardware did not distort the readings a number of long term tests were done. These showed no long term drift effects over periods of several days. Checks were also made to ensure that there was no cross-linking between the load cell and the potentiometer channels.

## 2.5 ERRORS AND APPROXIMATIONS

### 2.5.1 Non-Uniform Strain Distribution

The ring shear apparatus does not provide a uniform distribution of shear strain. The soil near the outside perimeter of the sample is sheared considerably more than the soil near the inside perimeter.

During the first few millimetres of shearing when the strength of the soil is strongly dependent on displacement, the error will be significant. One consequence is that the peak strength in the ring shear apparatus will be less than that measured in the direct shear apparatus. After the first few millimetres of shearing the strength asymptotically approaches the residual strength and becomes independent of the displacement. This implies that the residual strength measurements will not be distorted by this effect.

The same geometric effect applies to the shearing velocity. The soil near the outside perimeter of the sample is being sheared at a faster rate than the soil on the inside perimeter. The assumption that the average strength over the range of velocities will be the same as that for a uniform velocity distribution depends on the strength/velocity function being linear over the respective velocity range. Noting from past researchers that the variation in strength is very small even over many orders of magnitude in velocity, this assumption is valid. Errors generated by the velocity distribution can thus be discounted.

### 2.5.2 Non-Uniform Normal Stress Distribution

It is commonly assumed that the normal load applied by the loading yoke is uniformly distributed. While force balancing dictates that the stress distribution about the perimeter of the sample must be uniform, variations in stress across the section of the sample are possible. With soil extruding about the edges of the sample during both the consolidation and shearing stages of the test, the normal stress is likely to fall off in the immediate vicinity of the upper platen edge.

Bishop et al (1971) undertook a thorough mathematical analysis of the possible effects a non-uniform normal stress distribution might have on the measured residual angle of friction. They concluded that for even the most unlikely normal stress distributions the effects are small and can be neglected.

### 2.5.3 Soil Friction in Gap

The narrow gap between the upper platen and the base rapidly fills with soil during the consolidation and shearing stages of the test. The process of soil loss and the effects on soil behaviour are discussed in the next chapter. This section will consider the effects of any friction forces that may develop in the soil filled gap and the effect they will have on the mechanical load measuring system.

The friction that develops in the gap will cause the strength of the soil to be overestimated as the moment caused by this friction will add to that caused by the strength of the soil. The magnitude of the error will depend on the strength properties of the extruded soil, the stresses present in the gap and the size of the gap. A precise calculation of the magnitude of this error is not possible as the stress regime in the gap is unknown, but likely to be complex. Only a crude approximation is possible.

It is assumed that the horizontal normal stress in the gap varies linearly from zero at the top of the lip (Point A in Figure 2.14) to a maximum at the failure surface (Point B). The maximum horizontal stress can be estimated from the known vertical stress and the coefficient of lateral earth pressure, $k_o$. For this crude analysis, $k_o$ is estimated to be 0.5 and thus the maximum horizontal stress will be half the effective stress ($\sigma'$). It is assumed that the stress variation is linear.

$$\sigma'_x = \frac{\sigma'(l-x)}{2\,l} \qquad \text{Eqn 2.13}$$

where $x$ = distance from top of the lip.

$\sigma'_x$ = effective stress of soil in gap in horizontal plane.

$l$ = depth of lip.

The product of the normal horizontal stress and the tangent of the residual angle of internal friction will give the shear stress about the surface of the upper platen. This can be integrated over the area of both the inside and outside lips to give the total frictional moment.

$$M_{f3} = 2\,\pi\,\mathrm{Tan}\,\phi_r\,(r_o^2+r_i^2)\int_0^l \sigma'_x\,dx \qquad \text{Eqn 2.14}$$

where $M_{f3}$ = moment error due to soil friction in gap.

Substituting Equation 2.13 into Equation 2.14 enables the integral to be evaluated.

$$M_{f3} = \pi\,\mathrm{Tan}\,\phi_r\,(r_o^2+r_i^2)\,\sigma'\,l \qquad \text{Eqn 2.15}$$

The error that this will cause in the measured stress can be determined as previously in Equation 2.9.

$$\delta\,\tau_{f3} = \frac{\mathrm{Tan}\,\phi_r\,(r_o^2+r_i^2)\,l\,\sigma'}{(r_o+r_i)\,(r_o^2-r_i^2)} \qquad \text{Eqn 2.16}$$

where $\delta\ \tau_{f3}$ = error in the shear stress as a consequence of soil friction in gap.

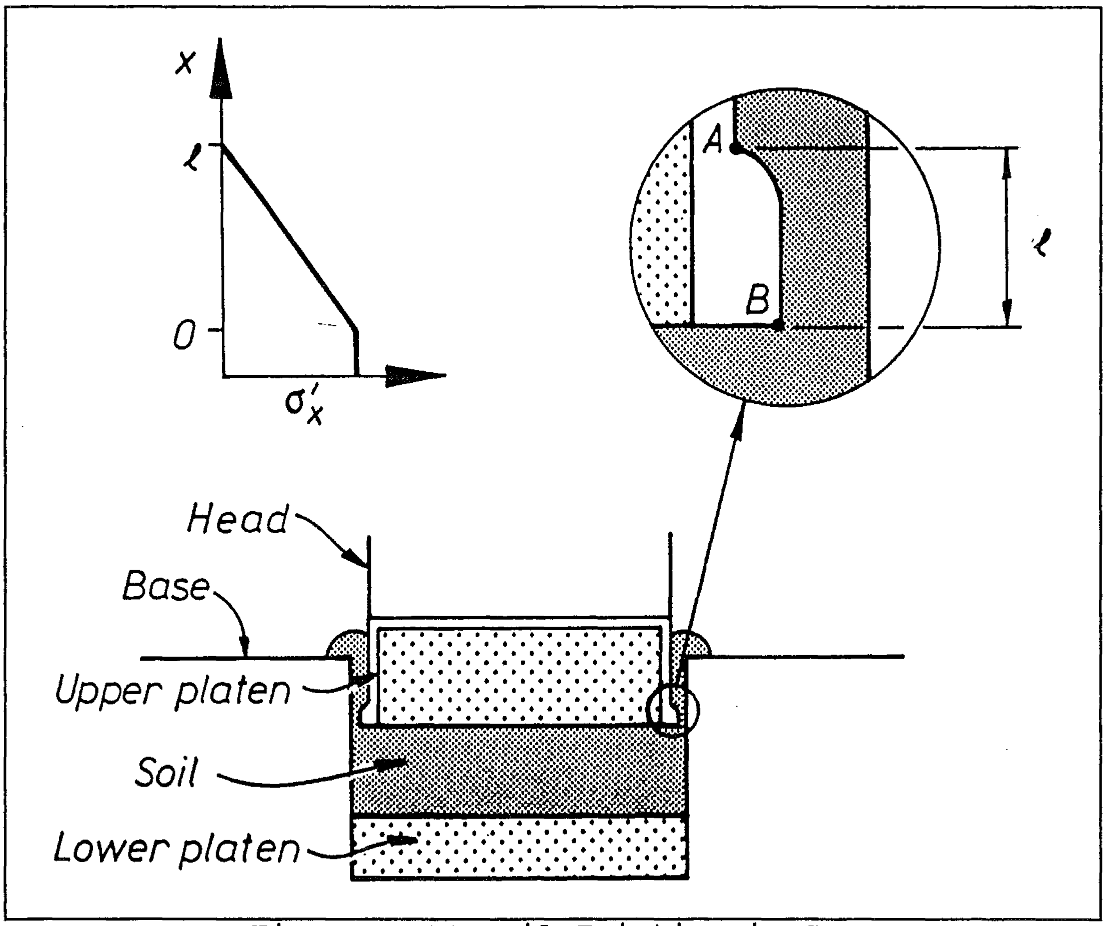



Figure 2.14 Soil Friction in Gap

Substituting the data for Test #1 into Equation 2.16 suggests that an error of 0.35 kPa (0.75%) in the measured shear stress. While there is considerable uncertainty in some of the assumptions made in this analysis, it nevertheless suggests that the magnitude of the error is tolerable. The analysis also suggests that this error is unlikely to be significantly altered by the rate of shearing.

### 2.5.4 Geometric Errors from Head Rotation

The determination of the shear strength from the load cell readings depends on the angle between the loading rod and arm being exactly perpendicular. When using the tertiary drive system, the head rotates and this angle is distorted. It was necessary to ensure that this does not result in any significant error.

At a maximum displacement of plus or minus 2.5 mm, the angle is distorted by 3.4° as illustrated in Figure 2.15. At this maximum displacement, trigonometry shows that the strength measurement is distorted by 0.2%. This is sufficiently small that a correction algorithm is not warranted.

### 2.5.5 Bearing Friction Errors

Three bearings in the modified apparatus may distort the measurement of the residual shear strength. The friction in the linear and needle roller bearing is analysed in Section 2.2.3 and quantified in Equation 2.10.

The thrust bearing between the loading yoke and head will produce a small moment in the direction of rotation. Equation 2.17 expresses the magnitude of this moment.

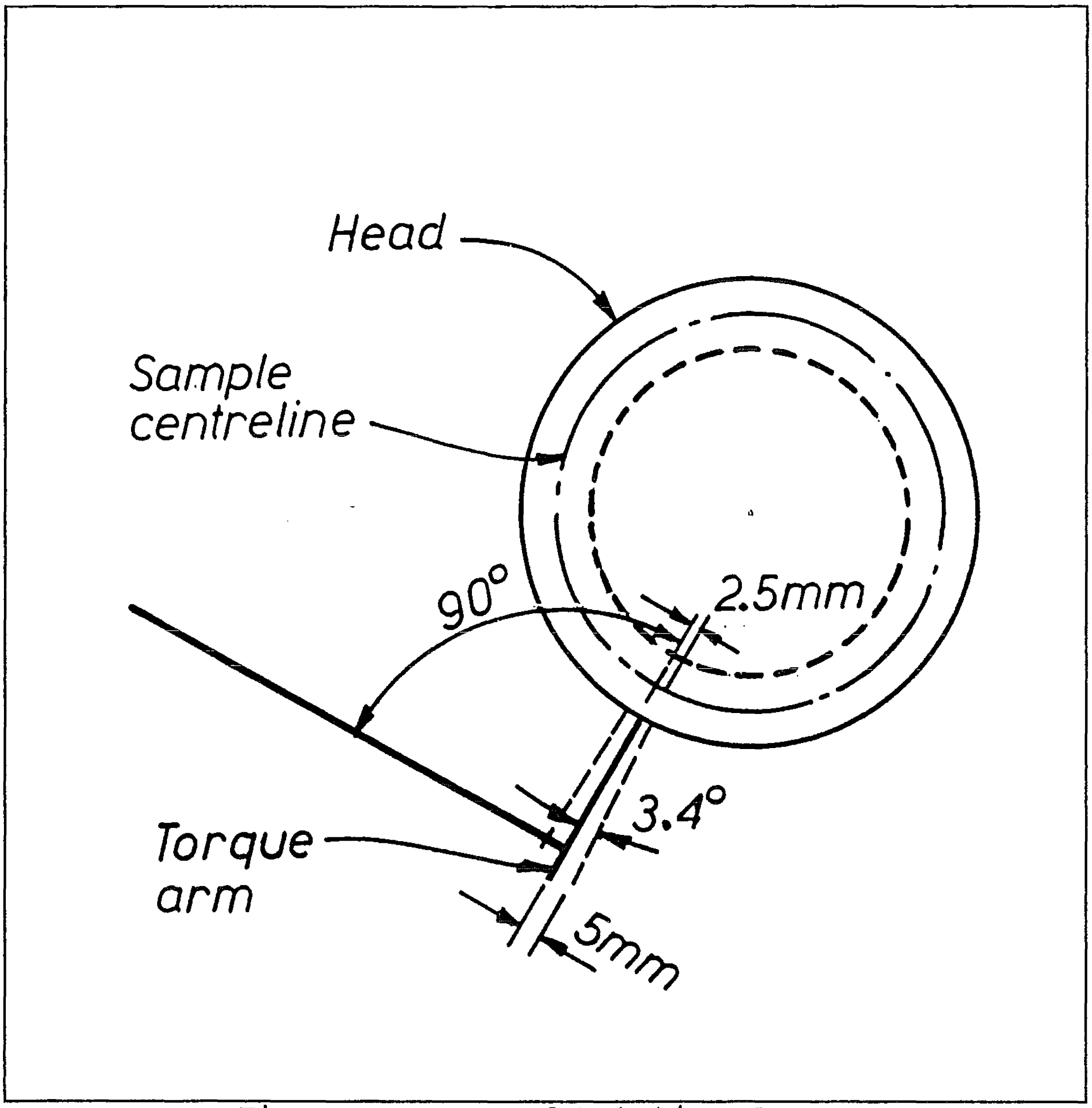



Figure 2.15 Head Rotation Errors

$$M_f = \mu_{tb}\ P\ r_{tb} \qquad \text{Eqn 2.17}$$

where $\mu_{tb}$ = Coefficient of friction for the thrust bearing.

$P$ = Normal Load on Sample.

$r_{tb}$ = Radius of Thrust Bearing

Combining Equation 2.17 into Equation 2.9 enables the error in the measured shear stress to be determined.

$$\delta\ \tau_{f4} = \frac{2\ \mu_{tb}\ \sigma\ r_{tb}}{(r_o + r_i)} \qquad \text{Eqn 2.18}$$

where $\delta\ \tau_{f4}$ = change in shear stress due to friction in thrust bearing.

The coefficient of friction for the thrust bearing is 0.0013 (SKF Bearing Catalogue 1975) and the radius of the bearing race is 6 mm. For Ring Shear Test #1 this equates to an error of 0.02 kPa in the measured residual strength.

It is particularly critical to this project that these errors are not rate dependent. This is not a problem for the thrust bearing as it is rotated at a constant rate by the small electric motor mounted on the loading yoke. The needle roller bearing is subject to movement rates as slow as 0.5 rev/year and as fast as 8 rev/min. However, bearing manufacturers studies show that at rates of less than 200 rev/min friction is not dependent on the rate (SKF Bearing Catalogue 1975).

### 2.5.6 Summary of Errors

Table 2.1 below shows the sources of error described in the preceding sections for a soil with a residual friction angle of 25°. The major source of error is the soil friction in the gap. However, no method of eliminating this error was devised.

The error analysis suggests that the residual shear strength determined by the modified apparatus would be overestimated by approximately 1.1%. This error is not expected to be dependent on the rate of shearing.

In determining the difference in strength at different shearing rates, errors from the load cell, data acquisition system, and the rotation of the head, must be considered. These total 0.25% and are suggested to be an approximate upper limit to the accuracy of any rate dependent measurements.

Table 2.1 Error Summary

| ERROR SOURCE | % ERROR IN MEASURED STRENGTH Overestimation | Underestimation |
|---|---|---|
| Load Cell (Section 2.3) | +0.025% | -0.025% |
| Data Recording System (Section 2.4) | +0.025% | -0.025% |
| Soil Friction in Gap (Equation 2.16) | +0.750% | 0% |
| Head Rotation (Section 2.5.4) | +0.200% | -0.200% |
| Linear and Needle Roller Bearing Friction (Equation 2.10) | +0.050% | 0% |
| Thrust Bearing Friction (Equation 2.18) | +0.020% | 0% |
| TOTAL ERROR RANGE | +1.070% | -0.250% |

In this chapter significant modifications to the Bromhead ring shear apparatus are described. These changes ensure that the residual shear strength can be measured accurately over as wide a range of shearing rates as possible. The chapter also examines the assumptions and errors associated with ring shear testing in the modified apparatus, in order to provide guidelines on the level of accuracy to be expected. With these points in mind, the apparatus can then be used to closely examine the behaviour of soil sheared at different shearing rates.

# CHAPTER 3

# SAMPLE LOSS AND THE GENERATION OF PORE PRESSURES

## 3.0 INTRODUCTION

The extrusion of soil about the perimeter of the ring shear testing apparatus requires careful study. The problem arises from the need to physically isolate the upper platen so that the shear load, measured as a moment, is due only to the strength of the soil. The gap created for this purpose inevitably allows the continuous extrusion of a small amount of soil.

Understanding the consequences of this soil loss is particularly important in applying ring shear test results to practical engineering applications. Due to the differences of scale and geometry, soil extrusion in the field landslide will have a negligible effect on the average residual strength mobilised over the failure surface area. In contrast, extrusion may have very significant effects on the measured value of residual strength in ring shear tests.

This chapter will study the effects of soil extrusion on the measured residual strength in the ring shear apparatus.

## 3.1 LITERATURE REVIEW

The problem of soil loss in the ring shear apparatus has been recognised by most researchers but few have studied the problem in sufficient detail to eliminate the possibility that it may distort the measured residual strength.

In 1971 Imperial College and the Norwegian Geotechnical Institute developed a new ring shear apparatus that allowed the size of the gap to be controlled (Bishop et al, 1971). The effect of enlarging the gap was studied. It was concluded that providing the gap was sufficiently large to prevent metal on metal friction, the gap size had no effect on the measured residual strength at slow rates of shearing. The implications of the loss of soil producing a non-uniform stress distribution across the sample were also considered. An analysis concluded that errors from likely non-uniform stress distributions were insufficiently large to significantly distort the measured residual strength.

Lupini (1981), using the Imperial College apparatus, estimated the friction that would develop within the gap between the confining rings and the extruded soil. By assuming the vertical load in this vicinity is the same as that within the confined sample, and using the soils residual strength to find the friction coefficient, he determined that the maximum error was simply the ratio of the confining ring area to that of the total sample.

Tika, in her thesis on the rate effects of the residual strength, using the Imperial College apparatus, discusses possible sources of error from sample losses (Tika 1989). Testing a sample at a relatively fast rate, first closing the gap completely (during which time the shear stress in the soil can not be determined), and then reopening it, she notes no change in the strength. She thus concludes that the effects of soil extrusion at fast rates are negligible.

While the Imperial College ring shear apparatus has received some attention with respect to this problem, the Bromhead apparatus has seen very little. In introducing his new, simpler machine, Bromhead (1979) acknowledges that volume changes of the sample cannot be measured because these are inseparable from settlement readings caused by soil being squeezed out. Other consequences of soil loss are ignored.

A thorough analysis of the consequences of sample losses in both types of ring shear apparatus would seem necessary.

## 3.2 ANALYSIS OF EFFECTS OF SAMPLE DEGRADATION

### 3.2.1 Sample Degradation

Soil loss is not usually measured in the various ring shear apparatus, but rather the change in height of the soil sample. Changes in the height of the sample will be due to both changes in voids within the sample and the loss of soil. In practise it is impossible to separate the measurement of the two.

The sample degradation rate $d_r$, can be defined as the change in sample height per unit displacement. This dimensionless quantity will be expressed in terms of mm/m so as to keep the numerical values within a convenient range. Factors that could be expected to influence the degradation rate include displacement, rate of shearing, size of the gap, normal stress, overconsolidation ratio, type of apparatus and type of soil.

Ring shear tests at various rates of shearing and normal stresses were performed on Temuka Clay in the modified Bromhead apparatus. Details of these tests are provided in Appendix IV. Continuous monitoring of sample depth during testing enables the degradation rate to be computed. The degradation rate for these tests is presented in Figure 3.1.

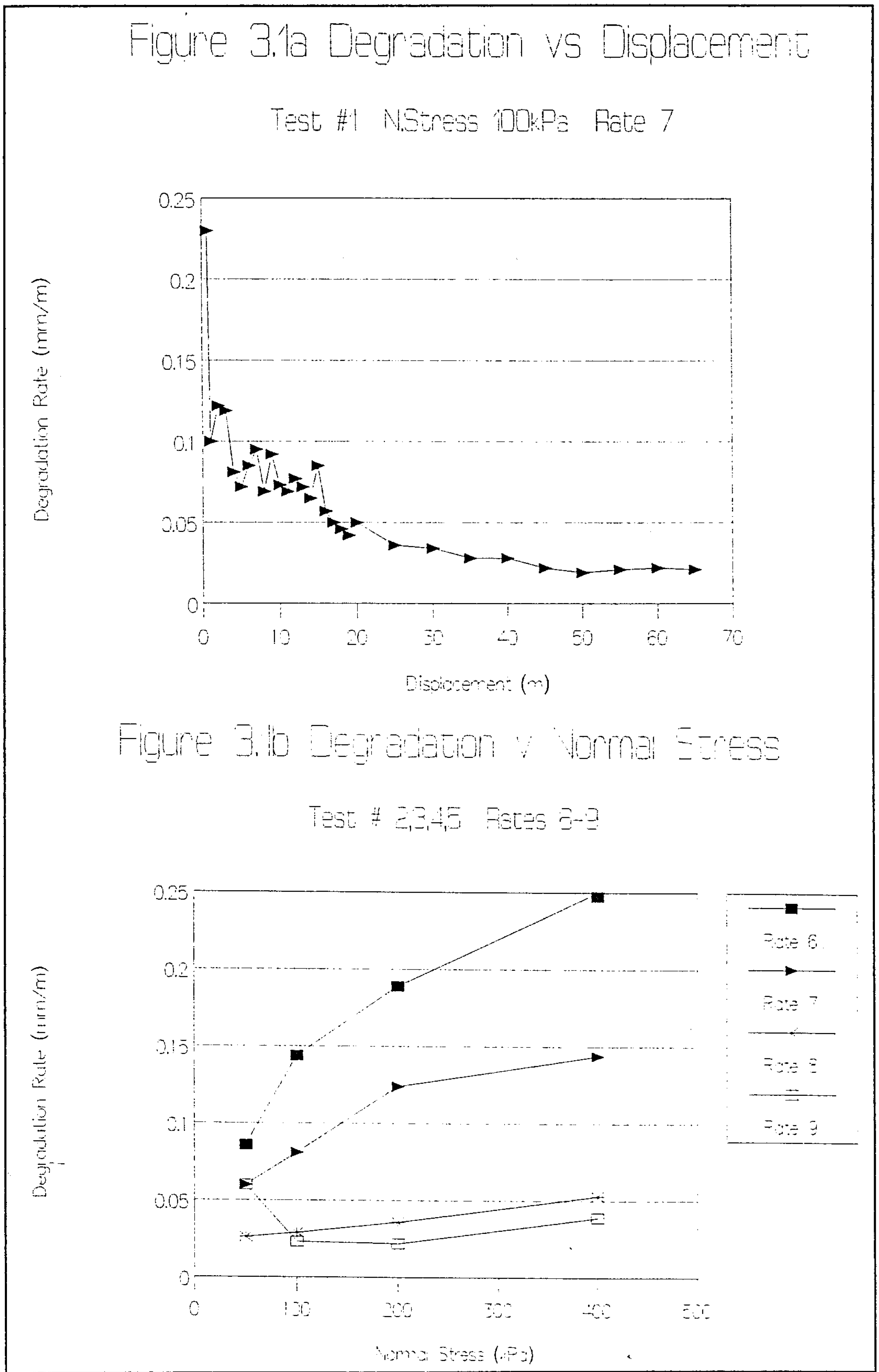



Figure 3.1 Factors Influencing the Degradation Rate

Sample loss is sufficiently small as not to significantly limit the magnitude of the displacements possible in the apparatus. In Test #1, the sample was sheared to a total displacement of 70m. The sample decreased in depth from 4.5mm to 1.6mm. As the residual strength is well established after a displacement of 5m, there is ample displacement remaining in which to study the effects of shearing rate changes on the residual strength.

It is apparent from Figure 3.1a that the degradation rate is greater during initial shearing and rapidly declines. This is expected as the degradation rate includes both soil losses and changes in soil volume. The initial increase in the shear loading will increase the total mean stress and cause the soil to consolidate. Consolidation tests detailed in Appendix II suggest that this process is largely completed 5 minutes after shearing is commenced. The degradation rate continues to reduce after this period suggesting that the loss of soil slows as the head sinks further into the base.

Figure 3.1b presents the data for Ring Shear Tests 2,3,4 and 5. Each test was conducted at a different normal stress and each sample was tested over a range of shearing rates. The data in this figure suggests that the degradation rate increases with increasing normal stress. At low stresses the changes in the degradation rate are less marked. The increased soil loss at larger normal stresses is expected. The greater normal stress would increase the stress gradient within the gap described in Section 2.5.3. This greater stress gradient would assist individual soil particles to overcome the frictional and gravitational forces and extrude through the narrow gap.

Data relating the shearing rate to the degradation rate, for the four fastest shearing rates, is presented in Figure 3.1b. (At very slow shearing rates the changes in the depth of the sample were too small to provide meaningful degradation rates.) The data for these fast rates indicates that the rate of degradation decreases with increasing rate of shearing. This trend requires further scrutiny.

Figure 3.1a shows a relationship between degradation rate and displacement. Ideally, to study the effects of varying the rate of shearing on the degradation rate, comparisons should be made at identical displacements. However, the testing schedule was designed primarily to study the residual shear strength, not the degradation rate, and the data presented in Figure 3.1b is at similar but not identical displacements. Comparisons between different rates of shearing at identical displacements and normal stresses are possible with Tests #1 and #2. These confirm the trend identified in Figure 3.1b.

Intuitively, it could be expected that when shearing at faster rates the degradation rate would decrease. The increased rate of movement gives soil particles less time to escape through the narrow gap in the apparatus.

It is important to recognise that the degradation rate is defined relative to displacement, not time. While the degradation rate falls with increases in the shearing rate, the loss of soil per unit time increases. This is because the range of shearing rates being tested is very large (from Rate 6 to Rate 9 represents a 300 fold increase), whereas the variation in the degradation rate is comparatively small (from 0.25 to 0.05 mm/m).

The trends indicated above are consistent with the data presented in Figure 3.1, with the exception of a single data point at the lowest stress of 50kPa and at the fastest shearing rate of 2.08 m/min (Rate 9). While it is possible that a different mechanism is being observed in this case, it is far more likely that some part of the apparatus or data collection system was failing to function correctly during this test.

It can be concluded from this section that the degradation rate increases with normal stress and decreases with respect to the displacement and shearing rate.

### 3.2.2 Water Demand

The extrusion of soil will cause the sample to demand or excrete water if the moisture content of the extruded soil varies from that of the remainder of the sample. In both the

Bromhead and Bishop type of ring shear apparatus, the soil extrudes from the sample in close proximity to the failure plane, and thus the moisture content of the extruding soil is likely to be at or very close to its critical moisture content (Atkinson & Bransby, 1978). If this is different from the moisture content of the remaining sample (usually the critical state moisture content will be greater), then a flow of pore fluid into the sample is necessary to maintain mass conservation. This can be quantitatively analysed by considering the conservation of pore fluid mass within the sample during residual shearing (ie. after sufficient displacement to form a stable shearing layer).

Figure 3.2 depicts the moisture content profile within the ring shear sample after sufficient shearing has taken place to form a stable shearing zone. The bulk of the sample will be at the initial moisture content, $w_i$, which existed prior to shearing commencing. The thin band of soil at the shearing surface will be at the critical state moisture content, $w_c$. Between these two regions there will be a transitional zone of intermediate moisture content.

As shearing commences and soil is extruded, the moisture content profile will move down the sample. Soil in the transitional zone will move into the shearing zone and undisturbed soil will move into the transitional zone. If $w_c > w_i$, as is normally the case, the soil in the transitional zone will dilate and in so doing will demand water. Alterna-

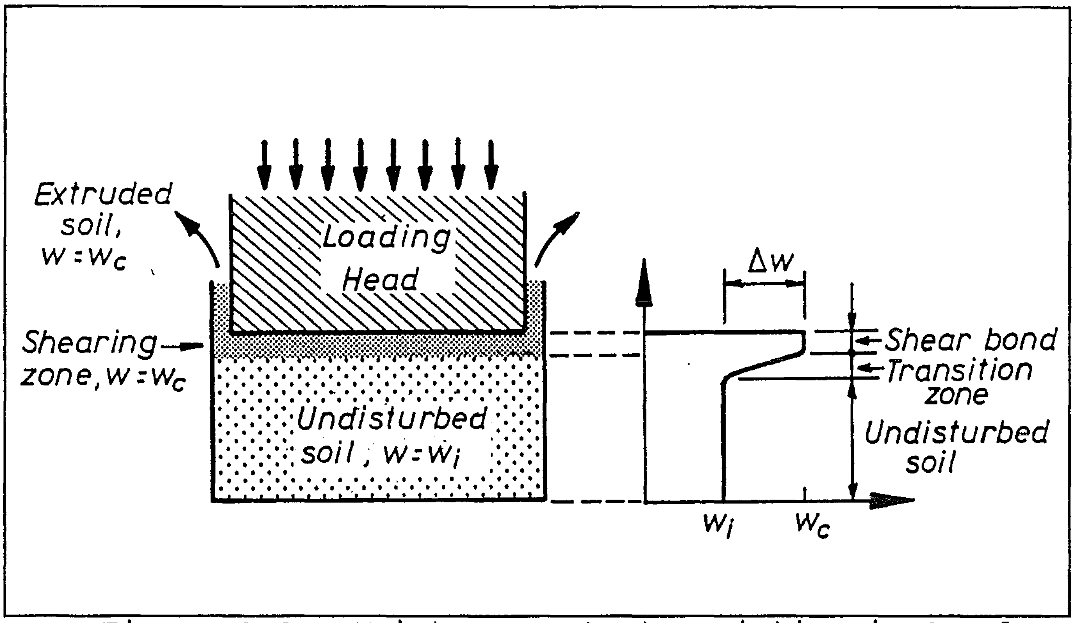


Figure 3.2 : Moisture Content Variation in Sample

tively, if $w_i > w_c$ , this soil will compress and consolidation will occur.

To determine the water demand, consider the change in depth of the sample $\Delta d$ over time $\Delta t$.

$$\Delta d = d_r \, v \, \Delta t \qquad \text{Eqn 3.1}$$

where v is the rate of shearing, and $d_r$ is the degradation rate defined in Section 3.2.1. Assuming the depth of the shearing band and transition zone remain constant, the depth of the undisturbed sample will reduce by $\Delta d$. The water demand can thus be calculated as the volume of water required to change the moisture content of a soil element of depth $\Delta d$, from its undisturbed to its critical state moisture content. The volume of the soil element will be :

$$V = A\, d_r\, v\, \Delta t \qquad \text{Eqn 3.2}$$

where A is the annular cross sectional area of the sample. Assuming the sample is saturated, and introducing G as the specific gravity of the solid particles, and $\rho_w$ as the density of water, the mass of solids in the element is :

$$M_s = \frac{A\, d_r\, v\, \Delta t\, \rho_w}{(w_i + 1/G)} \qquad \text{Eqn 3.3}$$

To change the moisture content of the soil element from $w_i$ to $w_c$ will take the following volume of water :

$$V_w = \frac{A\, d_r\, v\, \Delta t\, (w_c - w_i)}{(w_i + 1/G)} \qquad \text{Eqn 3.4}$$

Substituting $\Delta w = w_c - w_i$, the water demand caused by sample degradation is:

$$Q_D = \frac{V_w}{\Delta t} = \frac{A\, d_r\, v\, \Delta w}{(w_i + 1/G)} \qquad \text{Eqn 3.5}$$

Equation 3.5 shows that the water demand is proportional to the shearing rate and to the difference between the initial and critical state moisture contents.

To check the validity of the concept of water demand, the Bromhead ring shear apparatus was modified to enable the measurement of water entering or exiting the sample. Measuring the amount of water entering or leaving the lower platen of the apparatus was considered impractical because it continuously rotates during testing. Thus the lower drainage path was blocked off. The effect of having only one drainage path does not alter the fundamentals of the ring shear test, but simply increases the time required for pore pressures to dissipate.

The upper platen, which remains stationary, was connected to eight evenly spaced pipes which then joined through a manifold to an accurate burette. The burette was mounted on a height adjustable arm. As the burette reading changed the vertical position of the burette could be adjusted to ensure that zero head existed at the drained Vyon interface. These modifications are described in detail in Section 2.2.5 and are illustrated in Figure 2.8.

Ring Shear Test #1 consisted of shearing a sample of over consolidated Temuka clay at a constant rate of shearing of 46mm/min (See Appendix IV for details of test).

The initial moisture content of the sample was 28.4% while the soil that was extruded had a moisture content of 41.0%. Using the actual sample degradation data and these moisture content values it is possible with Equation 3.5 to estimate the water demand theoretically. Figure 3.3 shows both the calculated and the measured water demand rates for Test #1.

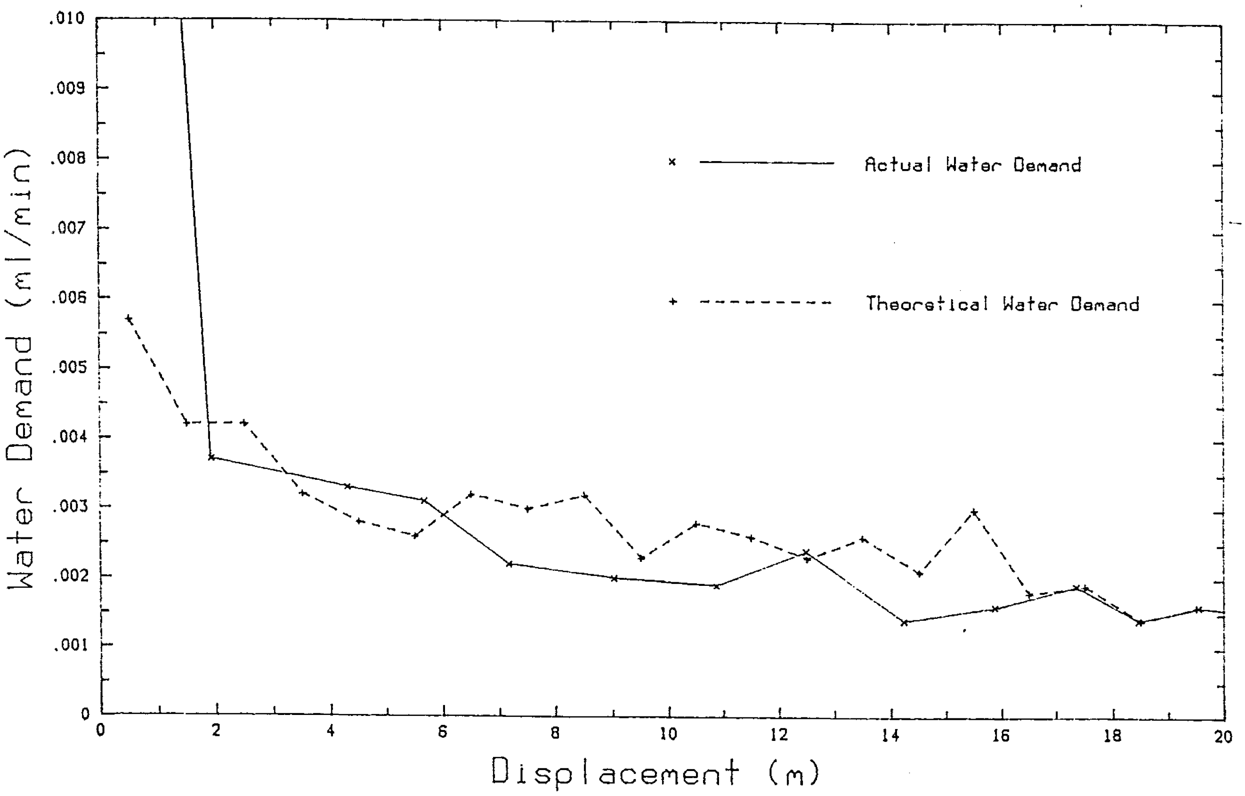



Figure 3.3 Comparison of Theoretical and Actual Water Demand

With both the magnitude and downward trend in the water demand being common to the theoretical and measured curves, the suggested theory seems to be valid. Only in the initial shearing phase do the two differ significantly. The far greater actual water demand in this interval is of no surprise as the theoretical model makes no account of the water demand required to form the initial shearing layer. That is, equation 3.5 holds only after a stable shearing zone is formed. The theory is evidently confirmed by these measurements.

Taking the measured water demand, and using the theory described, it is possible to determine an upper limit to the shearing zone's thickness. In the first 0.230m of displacement, the sample absorbed 0.14ml of water and reduced in depth by 0.079mm. The extruded soil, using equation 3.5, will require 0.055ml of water. This leaves 0.085ml of water to form the shearing and transitional zone shown in figure 3.2. It is not possible to determine either the depth or the moisture content profile of the transitional zone. However, if it is assumed that the transitional zone has zero thickness, then the shearing zone has a depth of approximately 0.11mm. A finite transition zone depth will reduce this value. This suggests that the 0.11mm is an upper limit to the shearing zone depth.

### 3.2.3 Pore Pressure Generation

Determination of the residual shear strength from the ring shear test is dependent on the assumption that no pore pressures exist within the soil sample. Critical state theory suggests that continued shearing will allow the soil within the region of large deformation to approach the critical state at which no further changes in void ratio will occur and no excess pore pressures will be generated. However this ignores the effects of continued water demand caused by soil extrusion as described previously.

The development of pore pressures due to water demand can be estimated by the application of Darcy's Law and the equation of continuity.

Even though the apparatus used in this research allows no flow through the lower platen, we will carry out a generalised derivation in which head losses will be considered within both the soil and the loading platens and both above and below the failure zone. Figure 3.4 diagrammatically represents the generalised problem geometry.

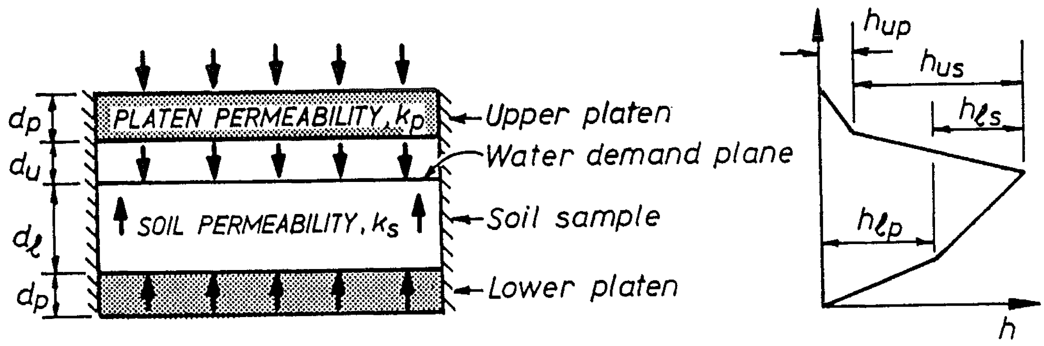



Figure 3.4 : Generalised Ring Shear Cross Section

Noting the geometry of a cross section of the ring shear sample and also noting that the side boundaries are impermeable, it seems reasonable to assume that flow takes place only in the vertical direction.

Some definition of the water demand plane is necessary. It is defined as the plane at which, for the purpose of determining pore pressures, the demand of water by the soil can be considered to be centred.

In the case of the smear type apparatus (ie Bromhead), this plane is positioned just below the shearing zone in the centre of the transition zone (see Figure 3.2). The moisture content profile in the split ring type apparatus (Bishop) will be similar except that it will have the same moisture content profile above the failure plane as below. Assuming the thickness of the shearing zone is small relative to the depth of the sample, the water demand plane can then be considered to be at the centre of the shearing zone in the Bishop apparatus.

The first step in the derivation is to consider the equation of mass conservation for the cross section :

$$Q_D = v_u A + v_l A \qquad \text{Eqn 3.6}$$

where $v_u$ and $v_l$ represent the pore fluid flux velocities to the upper and lower platens respectively. The use of Darcy's Law enables the head loss across each zone to be determined.

$$\Delta h_{up} = \frac{v_u \; d_p}{k_p} \qquad \text{Eqn 3.7}$$

$$\Delta h_{us} = \frac{v_u \; d_u}{k_s} \qquad \text{Eqn 3.8}$$

$$\Delta h_{ls} = \frac{v_l \; d_l}{k_s} \qquad \text{Eqn 3.9}$$

$$\Delta h_{lp} = \frac{v_l \; d_p}{k_p} \qquad \text{Eqn 3.10}$$

where the symbols in the above equation are defined in figure 3.4. Noting that the pore fluid pressure head at both the top surface of the upper platen and the bottom surface of the lower platen will be zero, continuity of pressure head at the water demand plane will require :

$$\Delta h_{up} + \Delta h_{us} = \Delta h_{ls} + \Delta h_{lp} \qquad \text{Eqn 3.11}$$

The pore water pressure at the water demand plane can be expressed as the pressure associated with the head defined by either side of equation 3.11 .

$$u = - \rho_w \, g \, ( \Delta h_{up} + \Delta h_{us} ) \qquad \text{Eqn 3.12}$$

where $\rho_w$ is the mass density of water and g the acceleration of gravity. Solving equations 3.6 to 3.12 to eliminate all the unknowns allows the pore pressure to be determined.

$$u = - \frac{\rho_w \, g \, Q_D \, T_c}{A} \qquad \text{Eqn 3.13}$$

where $T_c$ is the characteristic time defined by :

$$T_c^{-1} = \frac{k_s \, k_p}{d_1 \, k_p + d_p \, k_s} + \frac{k_s \, k_p}{d_p \, k_s + d_u \, k_p} \qquad \text{Eqn 3.14}$$

Substituting equation 3.5 into equation 3.13 allows the pore pressure to be determined directly from the degradation rate.

$$u = - \frac{\rho_w \, d_r \, g \, v \, \Delta w \, T_c}{(w_i + 1/G)} \qquad \text{Eqn 3.15}$$

Prior to looking at examples of how pore pressures can distort the measurement of the residual shear strength, it is important to recognise the differences between the various types of ring shear apparatus.

In the Bishop split ring type apparatus the sample is sheared at mid-depth and the sample is approximately 20mm thick. The porous stone platens, which are approximately 10mm thick, could be expected to be have a permeability of about $10^{-4}$ m/s, compared to $10^{-6}$ m/s in the case of the most permeable soil likely to be tested. Noting these values and equation 3.14 it is apparent that :

$$d_u k_p >> d_p \; k_s \qquad \text{Eqn 3.16}$$

and that $d_u \approx d_1$. This allows the characteristic time to be expressed more simply :

$$T_c^{\text{Bishop}} = \frac{d_u}{2 \; k_s} \qquad \text{Eqn 3.17}$$

In the case of the Bromhead apparatus, the drainage distance to the upper platen is much smaller. Results from Ring Shear Test #1 analysed in Section 3.3 suggest that the shearing zone is approximately 0.1mm thick. For the testing of impermeable soils ($k_s < 10^{-8}$ m/s), the inequality in equation 3.16 holds and the equation 3.14 can be simplified to :

$$T_c^{\text{Bromhead}} = \frac{d_u}{k_s} \qquad \text{Eqn 3.18}$$

If the inequality in equation 3.16 does not hold, as would be the case if the soil was quite permeable or the platens relatively impermeable, the more complex form of the characteristic time must be used. Note also that blocking the lower drainage path, for the purpose of measuring the water demand in the Bromhead apparatus, has no significant effect on the problem of pore pressure generation from sample degradation. This is because the water demand plane is so close to the upper platen that drainage to the lower platen is negligible.

The problem of pore pressure generation from sample loss can be compared in the two types of ring shear apparatus, using equations 3.17 and 3.18. In the Bishop apparatus $d_u$ = 10mm as compared to the Bromhead apparatus where $d_u$ is approximately 0.1mm. The consequence of the different drainage path distances is that the characteristic time, $T_c$, is approximately 50 times greater for the Bishop apparatus.

This suggests that in testing a soil at a particular shearing rate, the pore pressures generated will be approximately two orders of magnitude larger in the Bishop apparatus, assuming similar moisture content changes and degradation rates. If this assumption is confirmed by experimental data, it can be concluded that the Bromhead apparatus is more suitable for doing fast rate ring shear tests.

### 3.2.4 Limitations on Shearing Rates

The analysis in the previous section allows the pore pressures generated by sample degradation to be estimated. These pore pressures will distort the measurement of the residual shear strength by varying the effective normal stress, as expressed in equation 3.19.

$$\tau = ( \sigma - u ) \tan \phi_r \qquad \text{Eqn 3.19}$$

where $\tau$ = shear strength of soil

$\sigma$ = total normal stress

$u$ = pore pressure

$\phi_r$ = residual friction angle

The error in the measured residual shear strength, $\xi$, can thus be determined :

$$\xi = \frac{-u}{\sigma} \qquad \text{Eqn 3.20}$$

It would be a mistake to use the above analysis and equations to correct measurements of the residual shear strength. Too many variables are only approximately known. Critical factors in the analysis, such as the permeability of the soil and the depth of the failure zone, can only be crudely estimated. The analysis should instead be used for determining an upper limit to the permissable rates of shearing.

A maximum shearing rate , $v_{max}$, can be determined by substituting equation 3.20 into equation 3.15.

$$v_{max} = \frac{\sigma \, \xi \, ( w_i + 1 / G )}{\rho_w \, d_r \, g \, \Delta w \, T_c} \qquad \text{Eqn 3.21}$$

At shearing rates in excess of $v_{max}$ the generation of pore pressures from soil losses will introduce errors that distort the measured residual strength in excess of the acceptable error level $\xi$. Any value of $\xi$ may be nominated, but clearly, errors exceeding a few percent are unacceptable in a research effort to assess precisely the effects of shearing rate.

The concept of a maximum shearing rate at which measurements remain reliable has been considered before. Gibson and Henkel (1954) introduce the concept of a drained shearing rate for the direct shear test. This is defined as the rate of shearing that allows 95% dissipation of pore pressures in the time taken to reach the peak strength, and can be estimated from data collected during the consolidation phase.

The application of this limitation on testing beyond the initial period of shearing has no logical grounding.

Lupini (1981) suggests that the drained shearing rate for ring shear testing be defined as the rate at which 95% dissipation of pore pressures occurs within a displacement of 1mm. This definition is arbitrary; the 1mm distance is simply chosen as a convenient value and could equally have been 0.1mm or 10mm.

Neither of these methods provide an acceptable limit on the drained rate of shearing for the ring shear test. Equation 3.21 is superior to either of these methods as it provides a theoretically backed limitation on the rate of shearing.

## 3.3 IMPLICATIONS FOR RING SHEAR TESTING

### 3.3.1 Bromhead Apparatus

The practical implications of sample loss, and the subsequent generation of pore pressures, are best considered by example. Ring Shear Test #1 involved testing an over-consolidated silty clay at 100 kPa. Data for the following analysis is obtained from the relevant appendices.

$$d_u = 0.1 \text{ mm}$$

$$k_s = 4 \times 10^{-10} \text{ m/s}$$

$$\sigma = 100 \text{ kPa}$$

$$w_l = 28.4\%$$

$$w_c = 41.0\%$$

$$G = 2.65$$

$$\rho_w = 1000 \text{ kg/m}^3$$

$$d_r = 0.04 \text{ mm/m}$$

$$g = 9.81 \text{ m/s}^2$$

Substituting this data into equation 3.18 allows the characteristic time to be determined.

$$T_c = 2.5 \times 10^5 \text{ s}$$

To find the maximum rate of shearing, it is necessary to choose an acceptable level of error, $\xi$. As concluded in Section 2.5.6, the accuracy of the apparatus and the data acquisition system is limited to the range +1.07% to -0.25%. It would be reasonable to limit any errors from pore pressures to the average of these two values, say 0.7%. Substituting $\xi$ and the data above into equation 3.21 allows the maximum shearing rate to be estimated.

$$V_{max}^{Bromhead} = 2250 \text{ mm/min}$$

It should be emphasised that this upper limit is only an approximation. As noted earlier, in section 3.2.1, the degradation rate tends to fall with increasing shearing rate and displacement. This relationship has been ignored in this simplified analysis. When conducting fast shear tests close to the estimated maximum rate it is advisable to check the calculated value of $\xi$. Equation 3.22 below is derived from equation 3.21 and allows the level of error in the measured shear strength caused by pore pressure generation to be directly estimated.

$$\xi = \frac{\rho \ d_r \ g \ v \ \Delta w \ T_c}{\sigma \ ( \ w_i + 1/G \ )} \qquad \text{Eqn 3.22}$$

It is apparent from the analysis of Ring Shear Test #1 that the only tests in this research project that may generate significant pore pressures are those at the fastest rate of 2079mm/min (Rate 9). Table 3.1 below sets out the level of error for Ring Shear Tests 2,3,4 and 5 at the fastest rate, using the degradation rate measured during testing.

Table 3.1 - Errors caused by Pore Pressure Generation in Fast Rate Ring Shear Tests

| RING SHEAR TEST # | DEGRADATION RATE @ Rate 9 | % ERROR @ Rate 9 |
|---|---|---|
| 2 | 0.061 mm/m | 1.98 % |
| 3 | 0.0233 mm/m | 0.38 % |
| 4 | 0.0221 mm/m | 0.18 % |
| 5 | 0.0387 mm/m | 0.16 % |

It is apparent from Table 3.1 that errors introduced by the generation of pore pressures are sufficiently small in Ring Shear Tests 3, 4, and 5. This confirms that the study of rate effects will be accurate to within 0.7%. The analysis also indicates that the residual strength measurements obtained at the fastest shearing rate on Test #2 should be treated with caution. Concern was raised previously about the reliability of this data point in Section 3.2.1.

It is relevant to note that for soils of lesser permeability than Temuka Clay the limitations on the rate of shearing will be even more restrictive. The soil within the failure zone of the East Abbotsford landslide had a permeability of only $1 \times 10^{-11}$ m/s. Assuming similar degradation rates to those observed for Temuka Clay, tests could not be conducted with reasonable accuracy at rates greater than 50mm/min. Noting that the slide in the above case reached rates as high as 3000 mm/min, this significantly limits any ring shear studies of the soil behaviour of this landslide.

The permeability of the silty clay from the Clyde Slide in the Cromwell Gorge was measured as 1 x $10^{-10}$ m/s, indicating a maximum shearing rate in the Bromhead apparatus of 500mm/min. These maximums will be useful in chapter 5, in comparing the measured rate effects of the residual shear strength.

### 3.3.2 Bishop Apparatus

It has already been noted that the Bishop ring shear apparatus is far more susceptible to the problem of pore pressure generation due to water demand. This is because the drainage path, and thus the characteristic time, is an order of magnitude greater than in the Bromhead apparatus.

This can be illustrated by substituting the permeability of Temuka Clay and the dimensions of the Bishop Apparatus into equation 3.17.

$$k_s = 4 \times 10^{-10} \text{ m/s}$$

$$d_u = 10\text{mm}$$

$$\Rightarrow \quad T_c = 12.5 \times 10^{6} \text{ s}$$

Assuming a degradation rate, normal stress and error margin similar to those used in the analysis of Ring Shear Test #1 above, the maximum shearing rate can be determined from equation 3.21.

$$V_{max}^{Bishop} = 45 \text{ mm/min}$$

Ideally, the degradation rate data should be obtained from actual tests in the Bishop apparatus using Temuka Clay. A comparison of degradation rates for soils of similar composition in the two types of apparatus suggests that soil losses in the Bishop apparatus are greater, and this would imply an even smaller maximum shearing rate. An exact comparison is not possible as there is no Bishop type apparatus in New Zealand.

Extensive laboratory testing has been conducted at Imperial College, London, using the Bishop apparatus to study the effects of varying the shearing rate on the residual shear strength. In an analysis of pore pressure, it is useful to consider data from these tests.

Lemos (1986) conducted a series of tests on soils from the Kalabagh Dam project in Pakistan. The samples were tested at rates up to 6200 mm/min and the results were used in assessing the stability of landslides under seismic loading.

Data listed below is taken from Test #1 of Lemos' Kalabagh Dam tests. The distance to the failure plane, $d_u$, is determined by subtracting the consolidation during loading from the initial sample depth. All other data is obtained directly from the appendices and Table 5.1 (p.188) of Lemos' thesis.

$$\sigma = 211 \text{ kPa}$$
$$w_i = 27.2 \%$$
$$w_c = 27.6 \%$$
$$G = 2.72$$
$$\rho_w = 1000 \text{ kg/m}^3$$
$$d_r = 0.324 \text{ mm/m}$$
$$g = 9.81 \text{ m/s}$$
$$d_u = 8.5 \text{ mm}$$
$$k_s = 9.4 \times 10^{-11} \text{ m/s}$$

Substituting this data into equation 3.17 enables the characteristic time to be determined.

$$T_c = 45 \times 10^6 \text{ s}$$

The maximum shearing rate can then be estimated using equation 3.21 and a permissable error level, $\xi$ of 1%.

$$V_{max}^{Bishop} = 140 \text{ mm/min}$$

During the testing of this particular sample, Lemos used shearing rates of 400 mm/min. He observed that the measured residual shear strength at this fast rate increased markedly and concluded that a structural change was taking place in the behaviour of the soil when sheared at fast rates. It seems likely that this observed increase in strength was due to negative pore pressures generated by sample degradation rather than any structural change in the shearing process.

Normally a soil sample will be over-consolidated prior to testing and the porosity, and thus the moisture content, will be greater in the shearing zone than in the intact sample. In these circumstances the pore pressure generated by water demand will be negative and will increase the measured residual shear strength. However, the opposite case is also possible.

If the soil is only normally consolidated, the critical state moisture content may be less than the initial moisture content of the sample (ie $w_c < w_i$ ). In these circumstances the soil will consolidate during testing and positive pore pressures will be generated. This will cause a reduction in the measured residual shear strength at fast shearing rates.

A drop in the measured residual shear strength is reported in some tests conducted by Lemos on the Kalabagh Dam soils. When Test #3, on Clayey Siltstone A, was sheared at 100, 300 and 700 mm/min, the strength decreased by 11%, 53%, and 58%

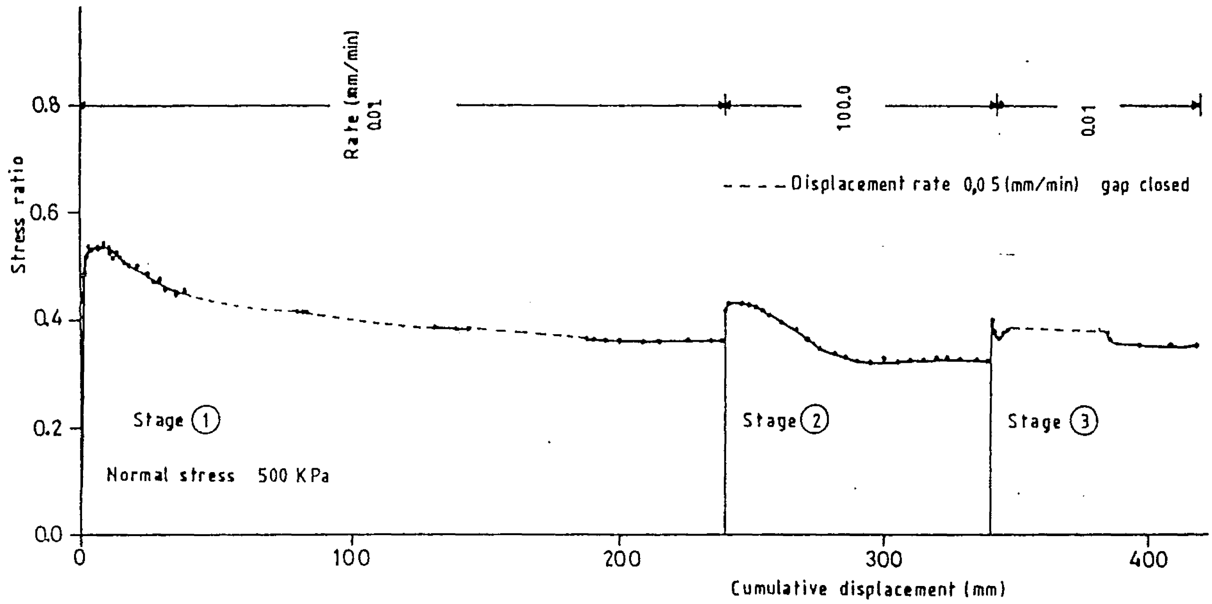
respectively. Lemos concluded that some structural change occurs in the shearing zone at fast rates which reduces the shear strength. This conclusion has very significant ramifications for assessment of seismic risk from landslides as well as the stability of creeping slopes.

Using the concept developed in this chapter, further inspection of the results observed by Lemos, leads to quite different conclusions. There are three ways in which the observed phenomena are qualitatively consistent with positive pore pressures generated by soil loss.

Firstly, the effect increases with shearing rate. Secondly, the measured moisture content in the shear zone is 0.5% less than the remainder of the sample indicating positive pore pressures and a drop in strength. Thirdly, inspection of the residual strength behaviour after the fast rate indicates that the phenomenon is pore pressure and not structurally based. The reported results of Lemos, reprinted in Figure 3.5, show that when, after fast shearing, shearing was stopped for a period in excess of the required consolidation time, and shearing was then recommenced at the slow rate, the strength immediately returned to its normal level. It is also interesting to note that the development of the low shear strength at fast rates does not occur instantaneously, but takes time to develop. This is also consistent with a pore pressure effect.

# KALABAGH DAM CLAYSTONE

## Sample 910 — Test #3



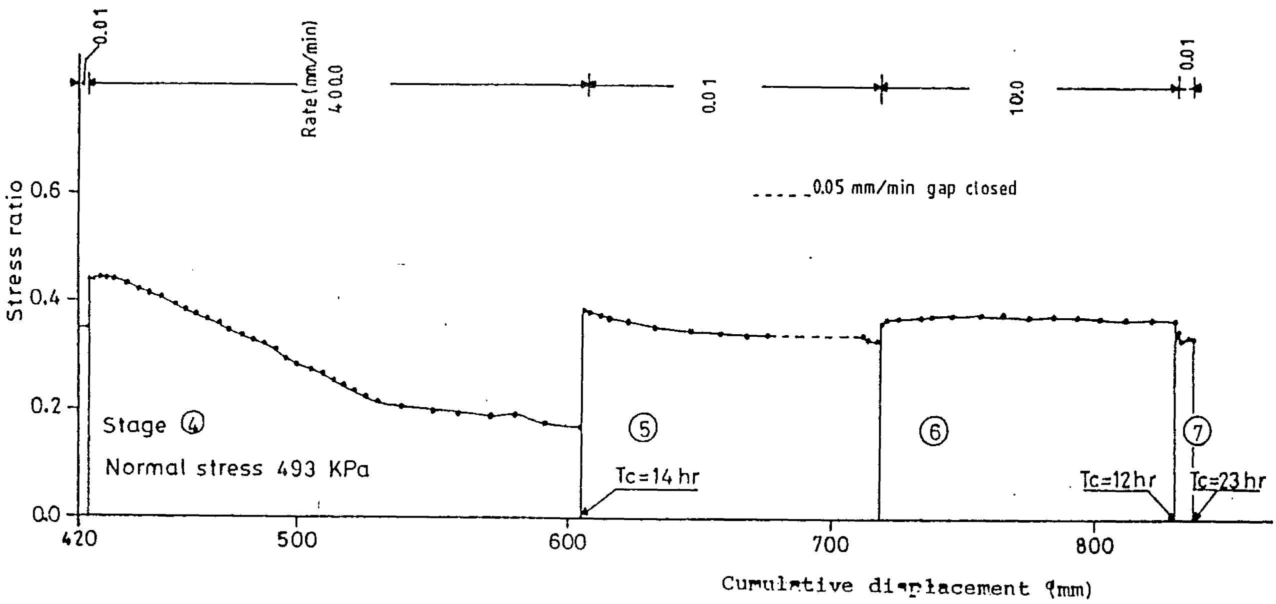




Figure 3.5 Ring Shear Test Results from Lemos (1986)

A final check that the results reported by Lemos for Test #3 are explained by pore pressures, generated by soil losses, would be to estimate, using equation 3.15, the magnitude of the pore pressures, and the resulting drop in strength, and compare this with the strength drop observed. The relevant data for Test #3 is listed below and, with equation 3.15, is used to determine the figures in Table 3.2.

$$\sigma = 200 \text{ kPa}$$

$$w_i = 20.5 \ \%$$

$$w_c = 20.0 \ \%$$

$$G = 2.66$$

$$\rho_w = 1000 \text{ kg/m}^3$$

$$d_r = 0.46 \text{ mm/m}$$

$$g = 9.81 \text{ m/s}$$

$$d_u = 9.1 \text{ mm}$$

$$k_s = 1.5 \times 10^{-11}$$

Table 3.2 Comparison of Theoretical and Actual Distortions in Kalabagh Dam Test #3

| Shearing Stage Number | Shearing Rate | Theoretical Pore Pressure | Theoretical Strength Distortion | Actual Strength Distortion |
|---|---|---|---|---|
| 11 | 100 mm/min | 18 kPa | - 9 % | - 28 % |
| 9 | 400 mm/min | 73 kPa | - 36 % | - 57 % |
| 15 | 700 mm/min | 127 kPa | - 58 % | - 60 % |

The magnitude of the estimated drop in strength is consistent with that observed in the Kalabagh Dam tests. The variation between the theoretical and actual distortion is acceptable, taking into account the level of accuracy of the data used in the sample degradation and water demand analysis, and the scatter of the measured strength at this fast rate. This confirms the view that pore pressures generated from soil losses can cause the residual shear strength to be either under or over estimated, depending upon whether the initial soil sample is drier or wetter than the critical moisture content.

## 3.4 CONCLUSIONS

The measurement of the residual shear strength in the ring shear test is dependent on there being no pore pressures present in the sample. It has been shown in this chapter that at fast rates of shearing, positive or negative pore pressures may be generated and thus may significantly distort the measurement of the residual shear strength.

This problem is found to be greatest in highly over consolidated soils of low permeability. The geometry of the testing apparatus is also a significant factor. The tolerances between the upper platen and the confining ring of the apparatus should be kept as small as possible so as to minimise soil losses and subsequently pore pressure generation.

The magnitude of pore pressure is proportional to the drainage path distance. As this distance is an order of magnitude less in the Bromhead apparatus than in the Bishop apparatus, it can be concluded that the Bromhead machine is the more suitable for studies of the residual strength at fast shearing rates.

Correcting for pore pressure is not considered a practical option. The variables are too difficult to accurately determine, and the analysis too approximate for the errors calculated to be anything more than crude estimates.

Alternatively, the magnitude of pore pressure could be measured. This has been attempted by Lemos (1986) and Tika (1989) but with only limited success. They observed a very wide scatter in their recorded data. Furthermore, it is very difficult to locate the pore pressure transducers in the shearing zone without distorting the measured residual shear strength and thus defeating the purpose.

To conclude, the rate of shearing must be limited to rates at which pore pressure generation due to sample degradation will have no significant effect on the measurement of the residual strength.

# CHAPTER 4

# ANALYSIS OF TEST RESULTS

## 4.0 INTRODUCTION

A key aim of this research work was to test soils at various rates in the ring shear apparatus. A large number of tests were carried out on a variety of soils. The majority of tests served to develop and improve the Bromhead apparatus, as described in Chapter 2, and contributed little directly to the understanding of soil behaviour. These modifications were necessary to allow the residual shear strength to be measured accurately over a very wide range of shearing rates. The series of tests actually used to study rate effects are detailed in Appendix IV, and are analysed in this chapter.

The testing programme for this thesis was kept as simple as possible. Only one soil, Temuka Clay, was tested and care was taken during sample preparation to ensure consistency between tests. Each test sample was similarly over-consolidated and during all stages of shearing the normal stress was kept constant.

In order to check the performance of the apparatus at large displacements, and to establish standard behaviour, the shearing rate was not varied for the entire duration of the first test. In further tests to study rate effects, the schedule was designed to ensure that between every fast or slow rate stage, there was a stage at a standard shearing rate. Direct shear tests were also conducted on the same soil at the same normal stress levels to provide further confirmation of the tests' accuracy.

Test data from other researcher s' work is complex. The variation of normal stress, shearing rate and soil type between tests make it difficult to draw reliable conclusions. The relatively simple testing programme described in this chapter provides an extensive range of comparable data from which firm conclusions can be drawn regarding rate effects.

## 4.1 RING SHEAR LABORATORY PROCEDURE

The laboratory procedure was designed to ensure that the soil sample modelled, as closely as possible, the conditions in natural landslides, and that tests could be repeated with maximum consistency.

The procedures vary from those recommended by Bromhead in the Wykeham Farrance Ring Shear Operators Manual for two reasons. Firstly, a very high level of accuracy was required for these tests, and, secondly, there had been modifications to the ring shear machine.

Particular attention was given during all stages of testing to ensure that the soil sample remained fully saturated. This was to guard against the retention of small pockets of air that may alter the pore pressure response of the soil.

The procedures adopted are detailed in the following sections.

### 4.1.1 Sample Preparation

A sample of Temuka Clay was obtained from New Zealand Insulators Ltd and stored in a humid environment. Samples of 1kg were mixed with distilled water into a slurry and placed in a desiccator for two days to remove all air. The 1kg sample was then consolidated to 300 kPa in a 250 mm diameter oedometer. This produced fully saturated samples of a consistency allowing them to be easily kneaded into the ring shear apparatus.

The apparatus was prepared for testing by filling the drainage measurement system with de-aired water and saturating the upper and lower platens in the desiccator. The sample was kneaded into the annular cavity, with care being taken not to trap pockets of air. The soil was levelled off with the top of the base, the excess soil being used to determine the initial moisture content.

The upper platen was installed and the water bath filled. Drainage pipes from the manifold were connected to the upper platen and the loading arm put in position ready for the consolidation stages.

### 4.1.2 Consolidation

The consolidation process was planned carefully to minimise the extrusion of soil during loading. As each weight is added, soil extrudes through the gap. If the increase in normal stress is too great, less soil will remain for the shearing stages of the test. The result is that the maximum displacement at which the residual shear strength can be reliably measured, is reduced.

The extent of soil losses during consolidation can be studied using data from the drainage measuring system. The burette enables the volume of water exiting the sample to be estimated during consolidation. This enables the change in depth attributable to consolidation to be determined. When this is compared to the actual change in depth, the difference is equal to the amount of sample extruded. Data from Ring Shear Test #1 shows that 67% of the change in the depth during consolidation stages was due to soil extrusion, most of which occurred in the first few stages. With this is mind, a more gradual increase in the initial loading was adopted, as detailed in Appendix IV.

As a large proportion of the initial change in sample depth is due to soil extrusion, and not consolidation, it would be quite inaccurate to use the data from the loading stages to determine the consolidation characteristics, as suggested by the Wykeham Farrance manual. For the purposes of this research, the consolidation characteristics of the soil are determined by independent oedometer tests. From the data in Appendix II, and the soil depth, the 95% consolidation time is determined as 5 minutes. A minimum consolidation time of 10 minutes was chosen to ensure that at least 95% of pore pressures would dissipate prior to the next loading stage.

Any consolidation that occurs during the testing process will produce pore pressures that may distort the measurement of the residual shear strength. These pore pressures will only occur when the soil is initially loaded and can be minimised by over-consolidating the sample prior to shearing. All samples in this study were overconsolidated by a factor of three to minimise this problem.

### 4.1.3 Determination of the Residual Shear Strength

When sufficient time has elapsed to guarantee that all pore pressures from the consolidation stage have dissipated, shearing of the sample can commence. Bromhead (1979) recommended that a shearing plane be established by quickly shearing the sample manually, without collection of data. While this method may be satisfactory for routine laboratory tests, a different approach was adopted in this research.

All samples were initially sheared to a displacement of 10 m during which strength and displacement data was collected and recorded. Residual conditions were well established in all tests at this displacement. Residual shear strength could thus be accurately determined by averaging the last few metres of shearing.

In determining an appropriate shearing rate, time had to be taken into account. In order to compare the results with those for the direct shear apparatus, a similar rate would have been preferable. However, this would have involved weeks of testing. As indicated in Chapter 3, rates as high as 2250 mm/min could be used without generating excessive pore pressures due to sample degradation. A medium rate of 46 mm/min was chosen. During all shearing stages, the small electric motor driving the thrust bearing system, illustrated in Figure 2.11, was operating.

Measurements during the shearing process were collected continuously by the data acquisition system described in Chapter 2. During the first 10 m of shearing, the sample depth and shear strength data were collected at 2 mm intervals. Water demand measurements were read manually from the burette at each consolidation and shearing stage.

### 4.1.4 Study of Rate Effects

The modifications to the Bromhead machine described in Chapter 2 enabled tests at a wide range of shear rates to be performed, using the 3 different driving mechanisms. Shearing rates as slow as 0.4 mm/day and as fast as 4000 mm/min are possible. This represents a variation of seven orders of magnitude.

The first step in designing the testing programme was to identify the rates at which the samples would be tested. Using the primary and tertiary drives, the range of rates is limited to the discrete rates corresponding to the gears that are available. For fast rate tests using the secondary drive a continuous range of rates is available since a variable speed motor is used. The rates chosen for this research work are shown in Table 4.1, with the corresponding numbering system that will be used in the rate effect study. Rate 4 was chosen as the standard rate of shearing, and can be generated by either the primary or tertiary drive mechanisms.

Table 4.1 Shearing Rates Used in Study

| Shearing Rate Number | Displacement Rate | Drive Mechanism |
|---|---|---|
| 1 | 0.00029 mm/min | Tertiary E 45-45 |
| 2 | 0.0022 mm/min | Tertiary D 54-36 |
| 3 | 0.018 mm/min | Tertiary B 30-60 |
| 4 | 0.124 mm/min | Tertiary A 36-54<br>or Primary D 36-54 |
| 5 | 0.93 mm/min | Primary C 45-45 |
| 6 | 7.0 mm/min | Primary B 54-36 |
| 7 | 46 mm/min | Primary A 60-30 |
| 8 | 310 mm/min | Secondary 1.2 |
| 9 | 2079 mm/min | Secondary 4.6 |

The actual shearing rates within the annular ring shear sample vary from the inner to the outer perimeter. This means that both the sample displacement and shearing rate referred to in this thesis are averaged over the sample area. At the outer perimeter, the soil is being sheared at a rate

18% greater than the average, and similarly, the soil at the inner perimeter is being sheared at an equivalently lesser rate. The impact of this radial variation on the study of rate effects will be small, because the expected changes in the residual shear strength will be only a fraction of the change in the average shear rate.

Radial variation in shearing rate will have an effect on the measured strength at small displacements. During initial shearing, when the strength is displacement dependent, the peak strength observed in the ring shear apparatus will be less than in the direct shear apparatus. This is because, unlike the direct shear, all parts of the ring shear sample do not reach peak strength simultaneously, and as a consequence, the peak is diffused.

It is important to the testing programme that the shear load is maintained between stages. If shearing is temporarily stopped, creep within the apparatus allows the shear load to drop below the residual strength. When shearing is recommenced it may take some displacement to re-establish the residual shear strength. This was observed in Ring Shear Test #1, when shearing was stopped between stages, and the shear strength did not re-establish the former value until after 200 mm displacement (See Appendix IV, Test 1). At very slow shearing rates it took a considerable amount of time for the slack in the gears to be overcome and for shearing to commence. Therefore, to avoid a drop off in the shear load in between stages, an overlap period was devised, during which more than one shear drive is turned on.

The required overlap period could be determined, using a dial gauge and stop watch. During the overlap period, it is assumed that the faster shearing rate is not significantly affected by the slower rate. This assumption is justifiable because the differences in the shearing rates are so large.

The tertiary drive mechanism complicates the testing programme because the drive must be regularly reversed to maintain the perpendicular geometry of the loading arm. This requirement is explained in Section 2.2.7.

The testing programme was designed so that after each slow or fast rate stage, the shearing rate is returned to a standard value. This will assist in isolating any error or unusual soil behaviour.

The testing programme is detailed on the following page in Table 4.2. The ring shear tests in Appendix IV followed this programme as closely as possible, although unforeseen circumstances resulted in some variations. The testing programme consisted of four runs. The first established the residual shear strength, the second studied rate effects at medium rates (4, 5 & 6), the third at slow rates (1, 2, & 3), and the final run was to study rate effects at fast rates (7, 8 & 9).

Table 4.2a Testing Programme to Study Rate Effects

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 0:00 | Rate 7 ON | 0.000 | | | |
| | | | 1 | #7 46 | 217 |
| 2:40 | | 10.000 | | | |
| 2:59 | Rate 4t*FOR | | 2.1 | #7 46 | 20 |
| 3:00 | Rate 7 OFF | 10.920 | | | |
| | | | 2.2 | #4 .124 | 40 |
| 3:40 | Rate 6 ON | 10.925 | | | |
| 3:41 | Rate 4t REV | | | | |
| | | | 2.3 | #6 7.0 | 40 |
| 4:19 | Rate 4t FOR | | | | |
| 4:20 | Rate 6 OFF | 11.205 | | | |
| | | | 2.4 | #4 .124 | 40 |
| 5:00 | Rate 5 ON | 11.210 | | | |
| 5:01 | Rate 4t REV | | | | |
| | | | 2.5 | #5 0.93 | 40 |
| 5:39 | Rate 4t FOR | | | | |
| 5:40 | Rate 5 OFF | 11.247 | | | |
| | | | 2.6 | #4 .124 | 40 |
| 6:00 | Rate 4p ON | 11.252 | | | |
| 6:01 | Rate 4t OFF | | | | |
| 6:02 | Rate 3 REV | | 3.1 | #4 .124 | 20 |
| 6:18 | Rate 3 FOR | | | | |
| 6:20 | Rate 4p OFF | 11.255 | | | |
| | | | 3.2 | #3 .018 | 60 |
| 7:20 | Rate 4p ON | 11.256 | | | |
| 7:21 | Rate 3 REV | | | | |
| | | | 3.3 | #4 .124 | 40 |
| 7:55 | Rate 3 OFF | | | | |
| 7:56 | Rate 2 FOR | | | | |
| 8:00 | Rate 4p OFF | 11.261 | | | |
| | | | 3.4 | #2 .0022 | 180 |
| 11:00 | Rate 4p ON | 11.265 | | | |
| 11:01 | Rate 2 OFF | | | | |
| | | | 3.5 | #4 .124 | 40 |
| 11:10 | Rate 1 FOR | | | | |
| 11:40 | Rate 4p OFF | 11.270 | | | |
| | | | 3.6 | #1 .00029 | 740 |
| 24:00 | Rate 4p ON | 11.270 | | | |
| 24:01 | Rate 3 REV | | | | |
| | | | 3.7 | #4 .124 | 40 |

* t = tertiary, p = primary drive

Table 4.2b Testing Programme to Study Rate Effects

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 24:38 | Rate 3 OFF | | | | |
| 24:39 | Rate 4t FOR | | | | |
| 24:40 | Rate 4p OFF | 11.275 | | | |
| | | | 4.1 | #4 .124 | 20 |
| 25:00 | Rate 7 ON | 11.278 | | | |
| 25.01 | Rate 4t REV | | | | |
| | | | 4.2 | #7 46 | 20 |
| 25.19 | Rate 4t FOR | | | | |
| 25:20 | Rate 7 OFF | 12.198 | | | |
| | | | 4.3 | #4 .124 | 20 |
| 25:40 | Rate 8 ON | 12.200 | | | |
| 25:41 | Rate 4t REV | | | | |
| | | | 4.4 | #8 310 | 20 |
| 25:59 | Rate 4t FOR | | | | |
| 26:00 | Rate 8 OFF | 18.400 | | | |
| | | | 4.5 | #4 .124 | 20 |
| 26:20 | Rate 9 ON | 18.403 | | | |
| 26.21 | Rate 4t REV | | | | |
| | | | 4.6 | #9 2079 | 20 |
| | Rate 4t FOR | | | | |
| 26:40 | Rate 9 OFF | 59.983 | | | |
| | | | 4.7 | #4 .124 | 20 |
| 27:00 | Rate 4t OFF | 59.985 | | | |

The data acquisition programme was designed to collect readings from the sample continuously during each run to ensure there were no blind spots in the data record. Data was collected at rates of 30 times per minute for the medium rate study, 6 times per minute for the slow rate study, and 50 times per minute for the fast rate study. At these rates it was possible to store all the data from a particular run on a single floppy disk.

## 4.2 DIRECT SHEAR LABORATORY PROCEDURE

A standard direct shear box was used to obtain measurements of the shear strength for comparison with the results from the ring shear apparatus. The tests were conducted according to the manufacturer's guidelines (Wykeham Farrance Direct Shear Manual). The shear load was measured with the same load cell used in the ring shear test and displacements and changes in sample depth were measured by two potentiometers. The data was collected using the Burr-Brown Data Acquisition Board attached to an I.B.M. compatible P.C and stored on floppy disks.

Maximum compatibility between the samples tested in the two different apparatus was desirable. Sample depth in the direct shear test is normally 20 mm, while that in the ring shear is 6 mm; a significant difference. It was decided to modify the direct shear box to accommodate a thinner sample, by placing a 7 mm thick porous stone packer in the base of the direct shear box. This ensured that a 6 mm thick sample would be sheared at mid-depth ie 3 mm.

In all aspects of sample preparation, the same procedures were adopted as for the ring shear tests described in the previous section. The samples were overconsolidated by a factor of 3 in stages at 10 minute intervals as detailed for each test in Appendix III. Before shearing was commenced, the sample was given several hours to completely consolidate to make sure that no pore water pressures were present.

In determining an appropriate shearing rate for the direct shear tests, three factors had to be taken into account. Firstly, the rate of shear had to be consistent with the limitations devised by Gibson and Henkel (1954). Secondly, to allow comparisons, the chosen rate should be close to one of the nine shearing rates used in the ring shear. Thirdly, a rate had to be selected from those provided by the gear combinations on the direct shear machine.

Substituting the relevant data from Appendix II into the equations derived by Gibson and Henkel (1954), and assuming a peak strength at 2 mm displacement, enables the maximum shearing rate to be determined.

$$v_{max} = 0.82 \text{ mm/min}$$

A shearing rate of 0.9 mm/min was selected, which is similar to Rate 5 used in the ring shear testing work. A slower run of 0.12 mm (Rate 4) was also used to compare results.

The maximum displacement in the direct shear apparatus is 6 mm. At greater displacements, errors produced by the change in shear plane area become too large. The test must be reversed for larger displacements. A series of cycles back and forward allows the samples to be displaced up to 100 mm.

## 4.3 DISCUSSION OF RING AND DIRECT SHEAR TEST RESULTS

### 4.3.1 Comparison of Measurements of Strength

A comparison of the results from the direct and ring shear tests enables the properties of both the soil and the two apparatus to be better understood. It also allows the conclusions of previous researchers, on the advantages and disadvantages of the respective apparatus, to be reviewed.

Figure 4.1, on the following page, summarises the results of the direct shear tests from Appendix III. The first three points correspond to the strength at 1 mm, 3 mm and 5 mm displacement respectively, while the remaining points relate to each forward or reverse cycle. To check that the rate of shearing was not excessive, the last two runs in each direction were sheared at the slower speed of 0.12 mm/min. Details of the testing schedule are also in Appendix III. The shear strength in each of the four tests differed insignificantly, confirming that the rate was sufficiently slow so as to prevent the build up of pore pressures.

While the data in Figure 4.1 is plotted as though the sample was sheared in the one direction to a displacement of 100 mm, in reality the shearing consisted of 16 different runs in alternate directions. The errors caused by the alternating shearing are not known. In some of the tests the strength oscillated when switching between the forward and reverse

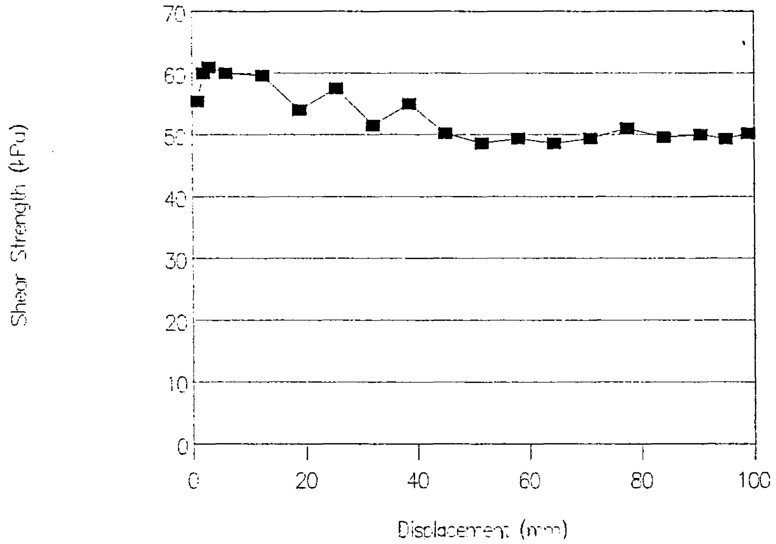



Figure 4.1a Direct Shear Test #1 100 kPa

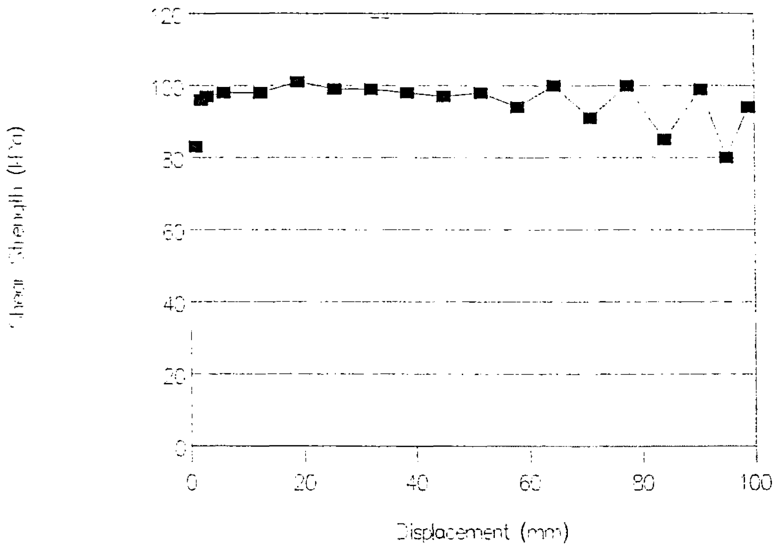



Figure 4.1b Direct Shear Test #2 200kPa

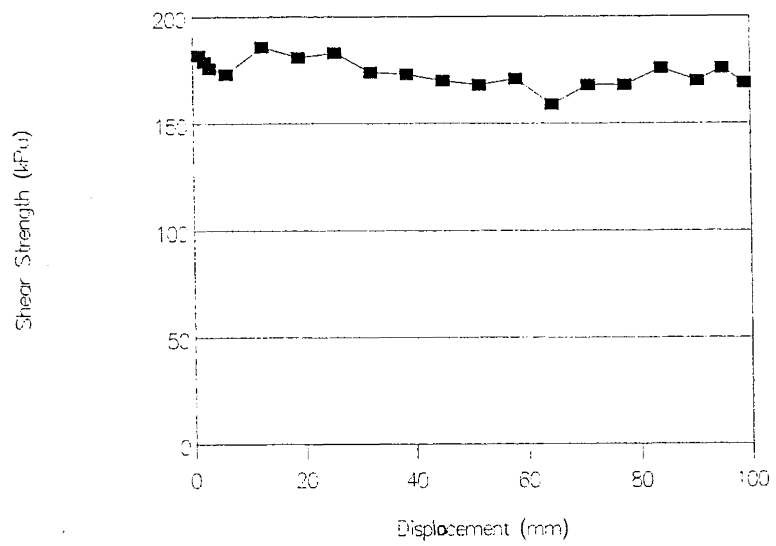



Figure 4.1c Direct Shear Test #3 400kPa

directions. This is particularly evident in Test #2. The reasons for this oscillation are likely to be related to distortions of the shearing zone caused by the reversals.

Before comparing results from the two types of tests, certain precautions should be borne in mind. At displacements less than 100 mm, the direct shear apparatus will produce satisfactory results. However, in this range the shear strength in the ring shear machine may be significantly distorted for two reasons. Firstly, the variation in the shearing displacement from the inner to the outer perimeter of the sample will reduce the magnitude of the peak strength, as explained in 4.1.4. Secondly, and more importantly, the shearing rate is such that insufficient time has lapsed at 100 mm displacement for pore pressures from the initial loading to disperse. It is possible to compare results with no pore pressures, and at the same normal stresses and shearing rate. However, with these results, it should be remembered that the direct shear samples have undergone a displacement of just less than 100 mm while the ring shear samples have been sheared just over 11 m. Results are presented in Table 4.3 on the following page.

The strength measured by the direct shear apparatus is consistently greater than that in the ring shear apparatus. This is expected. Bishop et el (1971) compared results from their ring shear apparatus with multiple reversals of the direct shear test on a number of soils. They found that the ring shear test consistently measured a lower strength.

Table 4.3 Comparison of Shear Strength in the Ring and Direct Shear Apparatus

| NORMAL STRESS | SHEAR STRENGTH (kPa) | |
|---|---|---|
| | Direct Tests | Ring Tests |
| 100 kPa | 49.7 kPa | 45.9 kPa |
| 200 kPa | 92.0 kPa | 86.2 kPa |
| 400 kPa | 173.0 kPa | 167.2 kPa |

The reason for the variation in the measured strength is best explained by Skempton (1985). He states that the post peak strength drop of an overconsolidated clay occurs in two stages. The first drop occurs at relatively small displacements, and is due to increases in moisture content as the soil approaches its critical state void ratio. The second drop in strength occurs over much larger displacements and is due to the realignment of the clay platelet particles parallel to the direction of shearing. This second process is disturbed in the reversals of the direct shear machine, leading to the observed greater strengths. The results observed in this work are consistent with this explanation.

4.3.2 Strength Reductions due to Displacements

A knowledge of the relationship between residual shear strength and displacement is useful. It assists the design of test procedures for ring shear testing and helps develop an understanding of landslide behaviour.

Figure 4.2 shows the results of Ring Shear Tests 2, 3, 4 and 5 for the first 5 m of displacement. The strength is presented as the ratio of the measured shear strength to the normal stress.

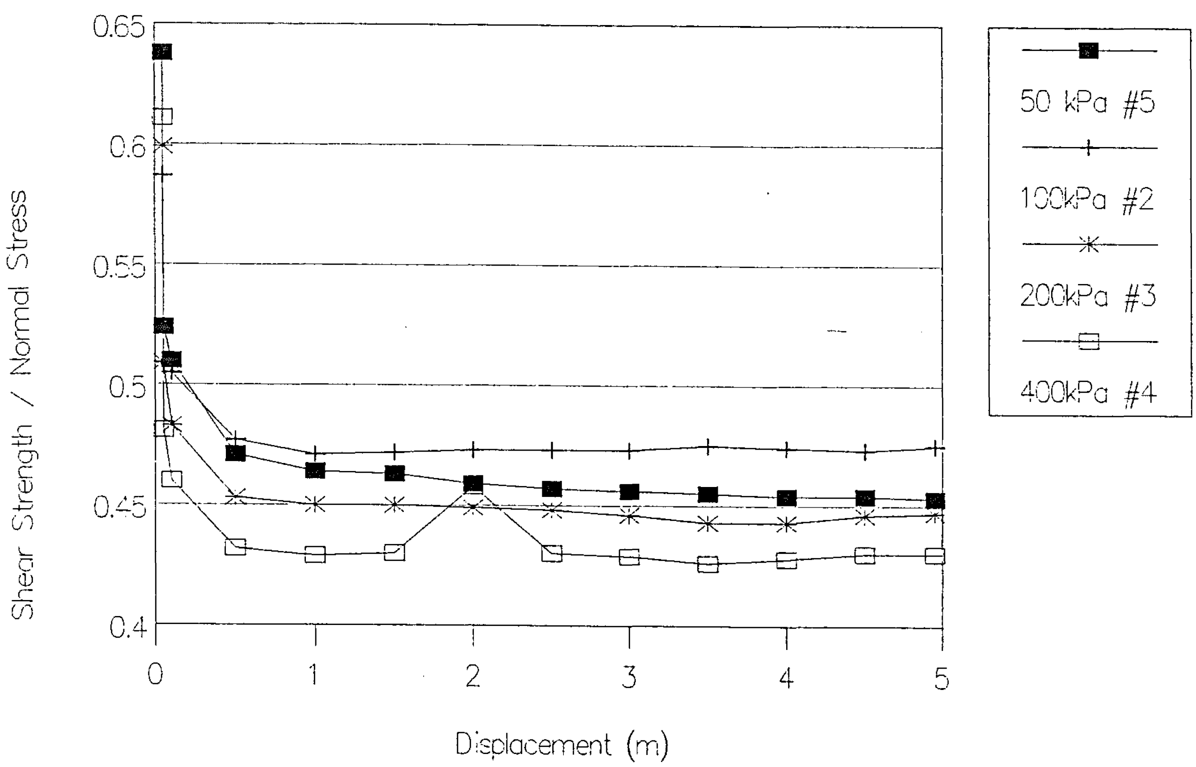



Figure 4.2 Development of the Residual Shear Strength

It is apparent from Figure 4.2 that residual shear strength is well established after a displacement of 5 m. The displacement required to achieve the residual shear strength appears to increase with decreasing normal stress. While in Test #4 (400kPa) the residual shear strength was well established by 1 m displacement, Test #5 (50kPa) needed almost 5 m to reach residual.

Results from the direct shear tests confirm this trend. In comparing the direct and ring shear test results, in Figure 4.3, it is noticeable that the variation in strength appears to be greatest at lower stresses. At 100 kPa the variation is 8.3%; at 200 kPa, 6.7%; and at 400 kPa, just 3.5%. With regard to the theoretical explanation of these differences, it would appear that the realignment of the clay particles occurs more quickly at higher stresses.

The recommended procedure for routine ring shear testing in the Bromhead apparatus provides for a displacement of 1 m before slow shear rates are used to measure residual shear strength. It appears from the results given here that this displacement may be insufficient for some samples at low normal stresses.

### 4.3.3 Determination of the Peak and Residual Friction Angle

Data from both the ring and direct shear tests described above are plotted in the Mohr diagram in Figure 4.3. For reasons explained in the previous section, peak strength data are taken from the direct shear test and residual strength data from the ring shear test. For consistency, the data plotted for the ring shear and direct shear tests are at the same rate of 0.9 mm/min (Rate 5).

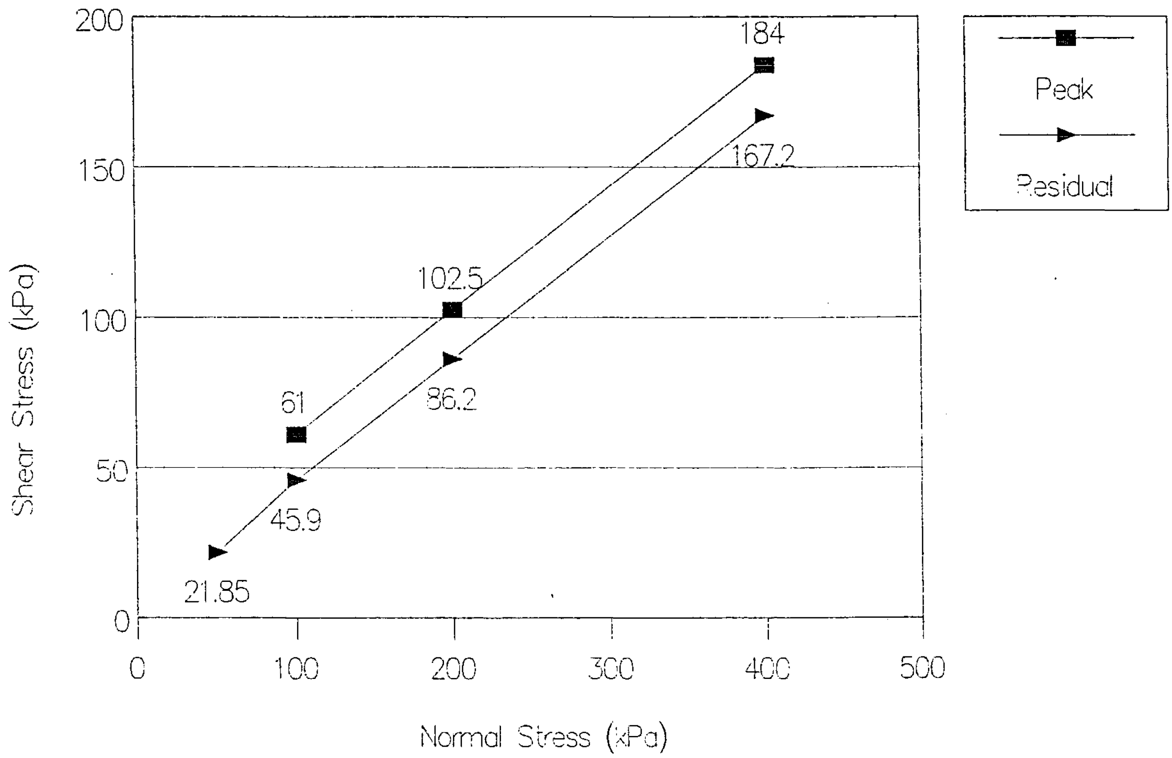



Figure 4.3 Mohr Diagram of Direct and Ring Shear Data

The friction angle can be determined with or without an apparent cohesion depending on whether the failure envelope passes through the origin. In the case of the residual strength in Figure 4.3, a line through the data points, determined by the least squares method, intercepts within 1 kPa of the origin. The residual friction angle is thus determined, with no cohesion, to be:

$$\phi_r = 23.4°$$

The peak strength results indicate an apparent cohesion. Using least squares method again, but allowing for an offset from the origin, allows the two peak strength parameters to be determined:

$$\phi_p = 22.4°$$

$$c_p = 19.9 \text{ kPa}$$

The close similarity of the friction angles suggests that the mechanisms that produce the difference in the peak and residual strength are not strongly dependent on the normal stress.

## 4.4 ANALYSIS OF RATE EFFECTS

Analysis of rate effects is complicated by the fact that changes in the residual shear strength are very small. Furthermore, even at a constant rate of shearing, the residual shear strength undergoes small random variations. The key to the analysis is being able to differentiate between natural variations and observed changes in the strength due to rate effects.

The problem is illustrated in Figure 4.4 which presents the raw residual strength data for Test #2 during the first four stages of the slow rate study. A statistical methodology had to be developed to be able to determine the difference in the shear strength at the different rates of shearing.

All data collected from the apparatus during testing and stored on floppy disk was processed using a computer programme. The programme scanned the data over pre-determined intervals for each of the channels. The use of a filter to eliminate bad points was tried, but found to be unnecessary. Averages and standard deviations determined for test runs 2 - 5 are provided in Appendix IV in Tables A4.2c, A4.3c, A4.4c and A4.5c respectively. The relevant statistics for Test #2 are also shown in Figure 4.4.

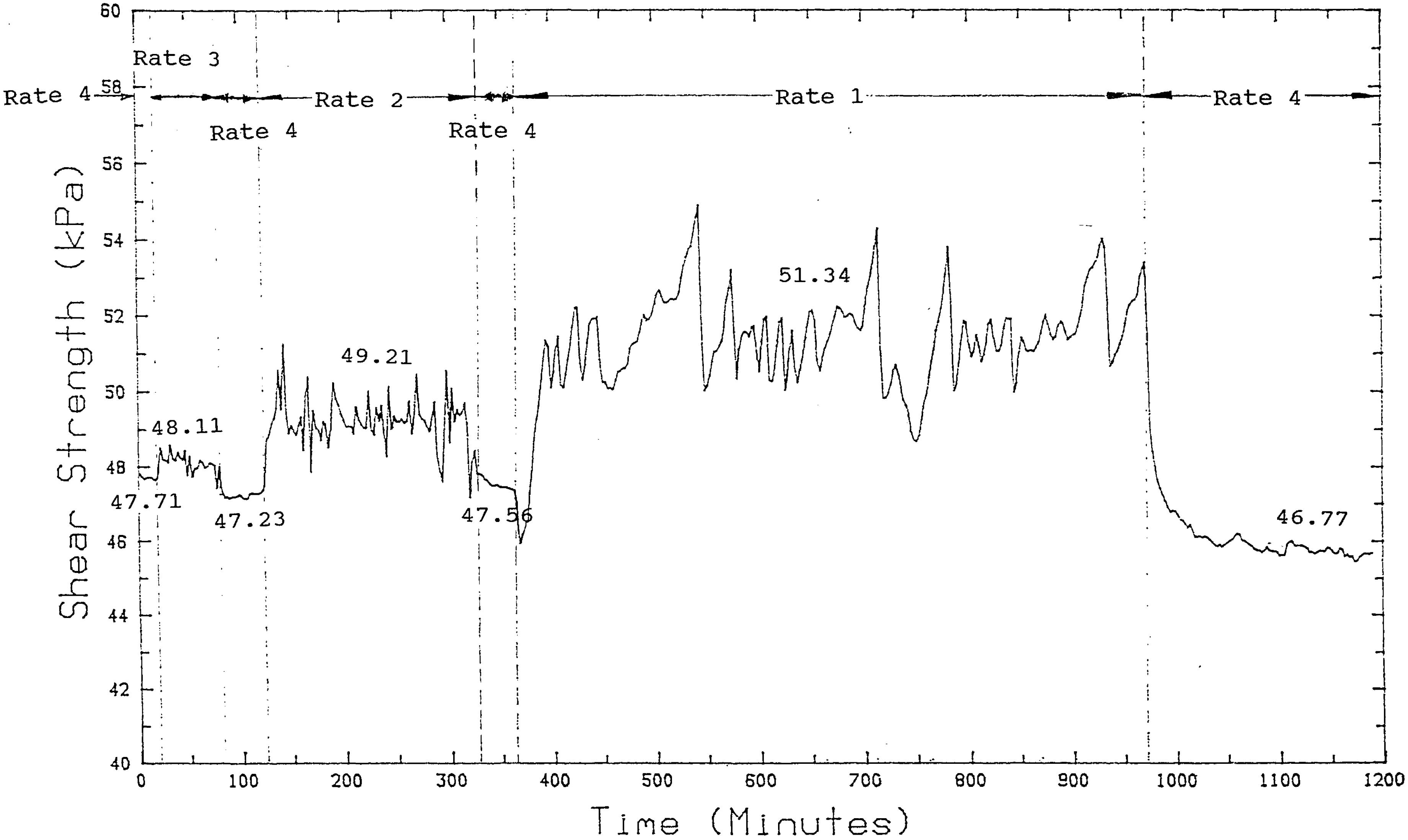



Figure 4.4 Example Data from Test #2 - Slow Rate Study

In order to measure the change in strength between a particular rate and the standard rate, the researcher must decide which strength value at the standard rate of shearing is to be used as the basis for comparison.

If variations in strength were completely independent of displacement, it would be admissible to simply average all the readings of the residual shear strength taken at the standard rate, and compare each other rate with this value. In Test #2, this would be the mean strength over many runs, including stages 1, 3 and 5. Alternatively, if a weak correlation exists between displacement and the measured strength, the strength at a particular rate should only be compared with the strength in the immediately preceding and following stages.

To determine whether strength is weakly correlated to displacement, a spectra analysis was undertaken. Each data series was extended out to the closest power of two with a cosine ramp, and then a standard fourier analysis programme was applied. The spectra for Test #1 are shown in Appendix IV. These reveal a distinct series of peaks, the largest being at a wavelength of 270 mm. This coincides with the average circumference of the sample, and suggests that the measured residual strength is weakly correlated to the angular position of the base.

These semi-regular variations in strength will not cause a problem in comparing strengths at fast rates of shearing. This is because the displacement over which the strength is averaged is many times greater than the circumference, so the calculated mean strength will not be significantly altered.

At slow testing rates, however, the displacements are only a small fraction of the circumference. This implies that comparisons of strength at slow rates may be distorted by the angular position of the base and that rate effects could be confused with regular variations in strength.

Solving this problem has been assisted by the testing schedule, which provides for shearing at the standard rate between each other rate. Rather than simply comparing the average residual shear strength at a particular rate with all other rates, it was decided to only compare it to the rate immediately preceding and following it. This ensures that any small variation as a consequence of the base position is not confused with rate effects.

### 4.4.1 Slow Rate Study

In Ring Shear Tests #2, #3, #4 and #5, there were variations in strength at slow rates. Results of these tests are illustrated in Table 4.4. To allow comparison of the results, the variations in strength between the standard rate

(rate 4) and the slow rates, and the standard deviation, are expressed as a percentage of the normal stress.

Table 4.4 Variations in the Residual Shear Strength at Slow Rates

| Test No | Normal Stress | % Variation in Strength from Standard Rate 4 | | | Standard Deviation of Shear Strength Ratio | | |
|---|---|---|---|---|---|---|---|
| | | Rate 1 | Rate 2 | Rate 3 | Rate 1 | Rate 2 | Rate 3 |
| 5 | 50kPa | +5.43% | +1.19% | +0.98% | 0.95 | 1.02 | 1.49 |
| 2 | 100kPa | +4.17% | +1.81% | +0.64% | 1.38 | 0.83 | 0.69 |
| 3 | 200kPa | +3.03% | +1.69% | +0.90% | 0.83 | 0.73 | 0.89 |
| 4 | 400kPa | +1.61% | +0.95% | +0.75% | 0.73 | 0.64 | 0.48 |

It is apparent from these results that, at all four normal stresses, the residual shear strength increases at slower shearing rates. The results also suggest, although less conclusively, that this increase in strength at slower rates is greatest at lower normal stresses.

The standard deviation of the shear strength at the various shearing rates and normal stresses indicates that differences in strength of less than 1% should not be given too much credence. It also appears, from the standard deviations, that the variability of the strength is greatest at low stresses and at the slowest shearing rates.

Close examination of the strength plots for each of the slow rate studies (in Appendix IV) indicates that the variability in the strength at the slower rates is semi-regular. The sharp periodical variations in the strength are likely to be due to the poor performance of the cog and worm gears at slow rates. During the many hours of testing at the slowest rate, a displacement of only 0.2 mm occurs. Even a small speck of dust in the gears may disrupt smooth movement at this rate. It is likely, at these very slow rates, that the actual rate of shearing may vary considerably about the average rate. However, even if the rate of shearing varies by as much as 100%, this will not significantly alter the study of rate effects. This is because the rates being studied range over many orders of magnitude.

### 4.4.2 Medium Rate Study

The variations in strength at medium shearing rates were studied in Ring Shear Tests #2, #3, #4 and #5. Results are analysed in Table 4.5. As in Table 4.4, the variations in strength at different rates, and the standard deviation, are expressed as a percentage of the normal stress, and rate 4 is taken as the standard rate. Where more than one test run was available to compare rates, the results were factored proportionally to the duration of the test to obtain the mean and standard deviation of the shear strength.

Table 4.5 Variations in the Residual Shear Strength at Medium Rates

| Test No | Normal Stress | % Variation in Strength from Standard Rate 4 | | | Standard Deviation of Shear Strength Ratio | | |
|---|---|---|---|---|---|---|---|
| | | Rate 4 | Rate 5 | Rate 6 | Rate 4 | Rate 5 | Rate 6 |
| 5 | 50kPa | +0.00% | -0.80% | -0.76% | 0.67 | 0.40 | 0.29 |
| 2 | 100kPa | +0.00% | -1.09% | +0.08% | 0.43 | 0.49 | 0.61 |
| 3 | 200kPa | +0.00% | -0.71% | +0.36% | 0.48 | 0.65 | 0.70 |
| 4 | 400kPa | +0.00% | -0.63% | +0.30% | 0.33 | 0.16 | 0.33 |

These results suggest a change in behaviour between Rate 5 and Rate 6. At Rate 5, all four tests are continuing to show a decrease in the residual shear strength at faster rates consistent with the trends observed in the slow rate study. The strength at Rate 6 is, in all four cases, greater than at Rate 5, indicating a change in behaviour. However, the variation at these rates is very small and is not significantly greater than the standard deviation. It will not be possible to conclude that strength gain is occurring until the fast rate results are analysed.

The variability in the results at these medium rates is less than at the slower rates, indicating that the gears in the loading mechanism are functioning more smoothly than at slower rates. It is also apparent from the medium rate study plots in Appendix IV that the new strengths at the different rates are very quickly established. Unlike at slow rates, it takes little time for the residual strength to stabilise.

Rate 4 is chosen as the standard rate against which all the other rates are compared. This shearing rate could be produced by either the primary or tertiary shearing drive. It is useful, as a check, to compare the results from the two drives. Comparing consecutive stages of each of the four tests with different mechanisms, but at the same rate, shows that the magnitude of the residual shear strength is consistent within 0.1%. However, it is interesting to note that the variation in the strength, as measured by the standard deviation, is greater in all four tests when the tertiary drive is used.

The data collected from the burette shows negligible water demand for all of the medium rate tests. This confirms that the measurement of the residual shear strength has not been distorted by pore pressure generation at these rates.

### 4.4.3 Fast Rate Study

Variation in strength at fast rates was studied in Ring Shear Tests #2, #3, #4 and #5. Results are analysed in Table 4.6. As in Table 4.4, variations in strength as a consequence of rate and, standard deviations, are expressed as a percentage of the normal stress. At shearing rate 7, more than one test run was available to compare rate effects, and so the results were factored proportionally to the duration of the test.

Table 4.6 Variations in the Residual Shear Strength at Fast Rates

| Test No | Normal Stress | % Variation in Strength from Standard Rate 4 | | | Standard Deviation of Shear Strength Ratio | | |
|---|---|---|---|---|---|---|---|
| | | Rate 7 | Rate 8 | Rate 9 | Rate 7 | Rate 8 | Rate 9 |
| 5 | 50kPa | +0.62% | +0.00% | -17.2% | 0.21 | 1.32 | 10.7 |
| 2 | 100kPa | +1.42% | +1.46% | +9.00% | 0.32 | 0.50 | 0.53 |
| 3 | 200kPa | -0.87% | +1.34% | +5.17% | 0.43 | 1.26 | 0.66 |
| 4 | 400kPa | +0.19% | +1.56% | +3.67% | 0.23 | 0.40 | 0.74 |

The study of rate effects on the residual shear strength of Temuka Clay at fast shearing speeds is less conclusive than the medium and slow rate studies. Generally, the data shows a continuation of the trend identified in the medium rate study, with the residual shear strength increasing with increased rates of shear.

It is apparent from the plots in Appendix IV it takes a finite displacement to re-establish a stable shearing strength after the fast rates. At slow and medium rates this was not so, and the strength re-establishes its former value immediately when the shear rate is returned to the standard rate. In the fast rate tests, the time required to re-establish the standard rate strength was approximately 15 minutes after rate 8, and 30 minutes following rate 9. The 95% consolidation time at this same stage was less than 3 minutes (because the sample thickness had reduced). This suggests that this behaviour was not due to pore pressures.

The above conclusion was confirmed upon analysis of the water demand data collected from the burette. Checks, using the actual water demand and equation 3.13, indicate that the negative pore pressures caused by sample degradation are sufficiently small to be ignored.

While the results here tend to indicate an increase in strength with increased shearing rate, there are two notable exceptions. Firstly, the data from Test #3 shows a drop in soil strength during run 4.2, when the shearing rate is varied from the standard rate to rate 7. The strength at the standard rate immediately preceding run 4.2 is unusually high when compared to other standard rate tests, but this alone is insufficient to explain the anomaly. Neither do the burette readings, which show negligible water demand, explain this behaviour. It can only be concluded that this result is inconsistent with the general trend but is not necessarily invalid.

Secondly, the behaviour of the soil in Test #5, at the lowest normal stress of 50kPa, showed some very unusual behaviour at fast rates. During run 4.4 at shearing rate 8, the strength of the soil varied little from the standard rate. At the other three stresses, the strength increased quite markedly.

Furthermore, during run 4.6 of Test #5, at shearing rate 9, the soil behaviour was even more unusual. The strength collapsed to only approximately 30% of the normal residual strength. This collapse in strength did not involve a smooth transition to a new strength. Rather, the strength fluctuated wildly and did not clearly re-establish a stable shearing strength at this fast speed.

When the standard shearing rate was restored, the strength gradually increased over a period of 30 minutes up to approximately its former value. The time taken at the standard rate to re-establish the former strength is significantly greater than the time required for any pore pressures to dissipate, indicating a structural rather than a pore pressure effect. This is confirmed by the burette readings which show a continuation of normal water demand behaviour.

Serious doubt is associated with the results from Test #5 due to problems experienced at the end of the preceding test. Test #4, at 400 kPa, had the largest degradation rate and during the last stage of shearing, at the very fast rate, the sample was completely extruded prior to the end of the test (see Run 4.6, Test 4 in Appendix IV). As a consequence, the roughened Vyon platens were sheared directly against each other for four minutes, which at that speed equates to 31 revolutions. It is possible that the roughened Vyon was damaged during this period. If so, the strength of the soil-platen interface may have been reduced to the extent that it was weaker than the internal strength of the soil. This would result in the failure plane forming at the interface, and a lesser strength being measured. It could also be assumed that the damaged platens may distort the measurement of rate effects during Test #5.

It cannot be assumed that this explanation of the unusual results in Test #5 is correct. However, the problem created by the over-run on Test #4, and the possible damage that this may have caused to the apparatus, is sufficient to raise question marks about the validity of the results from Test #5.

## 4.5 SUMMARY OF TEST RESULTS

Results of the slow, medium and fast rate studies of residual shear strength are summarized in Table 4.7 below. Shear strength data is presented as the ratio of shear strength to normal stress, where the shear strength has been normalised to that at the standard shearing rate. The residual shear strength ratio at the standard rate has been determined by a weighted average of the results for each test.

Table 4.7 - Summary of Rate Effect Study

| Shearing Rate (mm/min) | | Test #5 50 kPa | Test #2 100 kPa | Test #3 200 kPa | Test #4 400 kPa |
|---|---|---|---|---|---|
| 1 | 0.00029 | 0.4970 | 0.5131 | 0.4777 | 0.4446 |
| 2 | 0.0022 | 0.4546 | 0.4895 | 0.4643 | 0.4380 |
| 3 | 0.018 | 0.4525 | 0.4778 | 0.4564 | 0.4360 |
| 4 | 0.124 | 0.4427 | 0.4714 | 0.4474 | 0.4285 |
| 5 | 0.93 | 0.4347 | 0.4605 | 0.4403 | 0.4222 |
| 6 | 7.0 | 0.4351 | 0.4722 | 0.4510 | 0.4315 |
| 7 | 46 | 0.4489 | 0.4856 | 0.4387 | 0.4304 |
| 8 | 310 | 0.4427 | 0.4860 | 0.4608 | 0.4441 |
| 9 | 2079 | - | 0.5614 | 0.4991 | 0.4652 |

There is a high degree of confidence in the accuracy of these results with the exception of those from Test #5. As discussed in the previous section, there is concern that the strength may have been distorted by damage to the apparatus platens.

The strength-shearing rate relationship for the four tests is also shown graphically in Figure 4.5. In the following chapter, these results will be compared to those observed in other laboratory investigations.

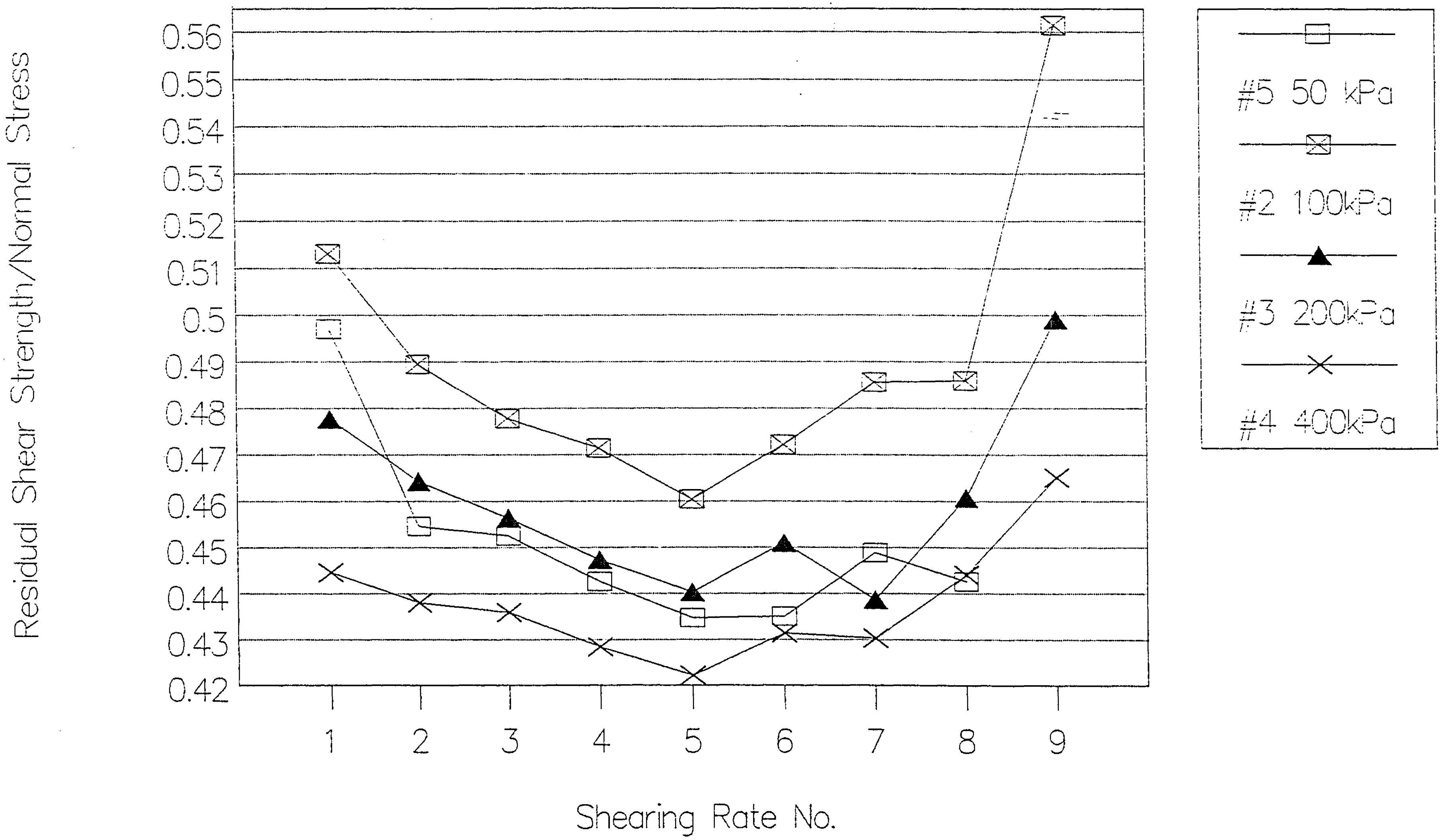



Figure 4.5 Summary of Rate Effect Study

# CHAPTER 5

# THE EFFECT OF RATE OF SHEAR ON THE RESIDUAL SHEAR STRENGTH

## 5.0 INTRODUCTION

A knowledge of the relationship between rate of shearing and residual strength allows the effects of external forces, such as rainfall events or earthquakes, on the stability of a landslide, to be determined. It has been emphasised previously that while the rate effects may be small, they will nevertheless be very significant in understanding landslide behaviour during failure.

In the previous chapter, the rate effects of one particular soil, Temuka Clay, were examined. Residual shear strength was found to vary by up to 15% over a wide range of shearing rates from as slow as 0.4 mm/day to as fast as 2 m/min. The change in strength was small in relation to changes in shearing rates, which ranged over seven orders of magnitude, confirming that the residual strength of a soil is not strongly dependent on the rate of shear.

In this chapter, the observed rate effects of Temuka Clay are compared with the results of other laboratory studies. In the final chapter, laboratory results will then be compared to observed field behaviour.

In discussing the rate effects observed by various researchers it is important to use consistent terminology. The first definition requiring clarification is the boundary between slow and fast rates of shearing. Skempton (1985) suggests a limit for slow rates of 1 mm/min, based on observations of changes in behaviour. This definition is used consistently by other Imperial College researchers. Salt (1985b) uses a similar definition of 1000 mm/day (0.7 mm/min). This thesis will use the 1 mm/min definition; rates in excess of this will be described as fast and rates below this figure termed slow.

A terminology has been developed to describe various rate effects. The most common terms are "positive rate effect", which applies when the strength increases with increasing shearing rate; and "negative rate effect" which applies when the strength decreases with increasing shearing rate. The same effects have also been termed "velocity strengthening" and "velocity weakening".

Some researchers have adopted different terminologies for rate effects, depending on whether the testing is at fast or slow rates. At fast rates the positive and negative terms are used. At slow rates an increase in strength with increasing shearing rate is described as the "conventional rate effect" and a decrease in strength with increasing shearing rate the "reverse rate effect".

It appears logical to use two different terminologies, as many soils, including Temuka Clay, behave quite differently at slow rates and fast rates. For this reason the "positive" and "negative rate effect" terms will be limited in this report to fast rates and the "conventional" and "reverse rate effect" terms will be used in relation to slow rates.

Studies of rate effects of soils are made more difficult by the fact that soil is a mixture of solid particles, liquid and gas. A knowledge of the shearing behaviour of all these components can assist in understanding the behaviour of soil.

The behaviour of liquids and gases in shear has been extensively studied in the discipline of fluid mechanics. The basic law governing the behaviour of fluids is Newton's law of viscosity:

$$\tau = \beta_a \ dv/dy \qquad \text{Eqn 5.1}$$

where $\tau$ = shear stress

$\beta_a$ = absolute viscosity

v = velocity

y = depth

Fluids which do not exhibit a constant viscosity are known as non-Newtonian fluids. The fluid components of soil : water and atmospheric gases, behave in a manner close to that of an ideal Newtonian fluid. Using the appropriate rate effect terminology, these soil components would exhibit both conventional and positive rate effects.

Solids behave in a more complex way. An elastic solid deforms proportional to shear stress until the yield stress is exceeded, and then deformation continues independently of the shearing rate. An ideal plastic (commonly known as a Bingham plastic material) behaves similarly, except that once the yield stress is exceeded, continued deformation occurs at a rate proportional to the stress in excess of the yield stress, as expressed in equation 5.2.

$$\tau = \tau_y + \beta \, dv/dy \qquad \text{Eqn 5.2}$$

where $\tau > \tau_y$

and $v$ = velocity of solid particles perpendicular to y axis

$y$ = vertical dimension in solid

The most commonly occurring materials do not behave as ideal plastic, in that ß does not remain constant. Most plastics exhibit "shear thinning", and ß decreases with increasing rate of shear. These substances are categorised as thixotropic plastics. If the opposite occurs and ß increases with increasing shearing rate, the substance is described as "shear strengthening".

Figure 5.1 illustrates graphically these different types of theoretical materials. It is important to note that all of these materials exhibit both conventional and positive rate effects, with the exception of the ideal fluid and solid. These substances exhibit no rate effects whatsoever.

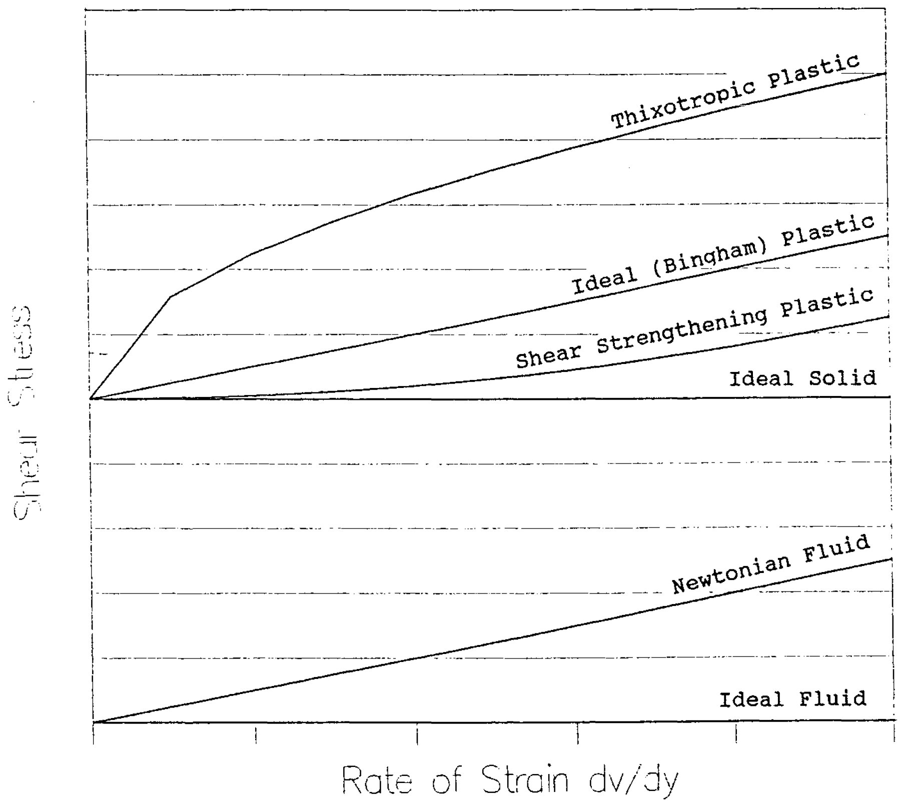



Figure 5.1 Theoretical Substance Definitions

These standard materials provide a bench mark with which to compare the observed behaviour of various substances in the laboratory.

## 5.1 STEEL

Soroushian and Choi (1987) report a series of extensive tests on structural steel, deformed bars and wire, in which the samples were subjected to monotonic tension at various strain rates. This study is principally concerned with residual strength behaviour, which in steel is analogous to behaviour after yield. Over a strain rate range of 0.00001 to 10 $s^{-1}$, they observed an increase in strength approximately proportional to the logarithm of the shearing rate. They derived the following constitutive model from their results:

$$\sigma_y = \sigma_y^* + \beta \log_{10} \dot{\epsilon}/\dot{\epsilon}^* \qquad \text{Eqn 5.3}$$

where
- $\sigma_y$ = tensile strength of steel in yield
- $\sigma_y^*$ = tensile strength of steel in yield at standard strain rate.
- $\beta$ = viscosity
- $\dot{\epsilon}$ = strain rate
- $\dot{\epsilon}^*$ = standard strain rate

The strength of the steel increased by an average 1.6% for each tenfold increase in strain rate. The behaviour of steel in tension is consistent with that of a thixotropic plastic, and exhibits conventional rate effects.

## 5.2 ROCKS

Laboratory studies of the rate effects for Granite, Dolomite, Halite and Quartz have been reported by a number of researchers. In this section, the results of these tests will be summarised to enable broad conclusions to be drawn.

### 5.2.1 Granite

Blanpied, Tullis and Weeks (1987) investigated rate effects on the steady state frictional strength of granite as part of their research into the stick-slip behaviour of faults. Annular samples of Westerly Granite were sheared in a ring shear apparatus at a normal stress of 50 MPa. The roughened surface was moistened with distilled water and, as testing progressed, approximately 0.1mm of gouge developed.

The shearing rate was varied from $6 \times 10^{-5}$ mm/min to 190 mm/min, a range similar to that used in the experimental work in this thesis. Figure 5.2 shows the steady state results measured by Blanpied et al.

Similar results on Westerly Granite are reported by Tullis and Weeks (1986). This study focused on the observed transient factors when the shearing rate was instantaneously stepped from one rate to another. The constitutive model developed by Rice and Gu (1983), and used by Tullis and Weeks to model the transient effects, is shown in equation 5.4.

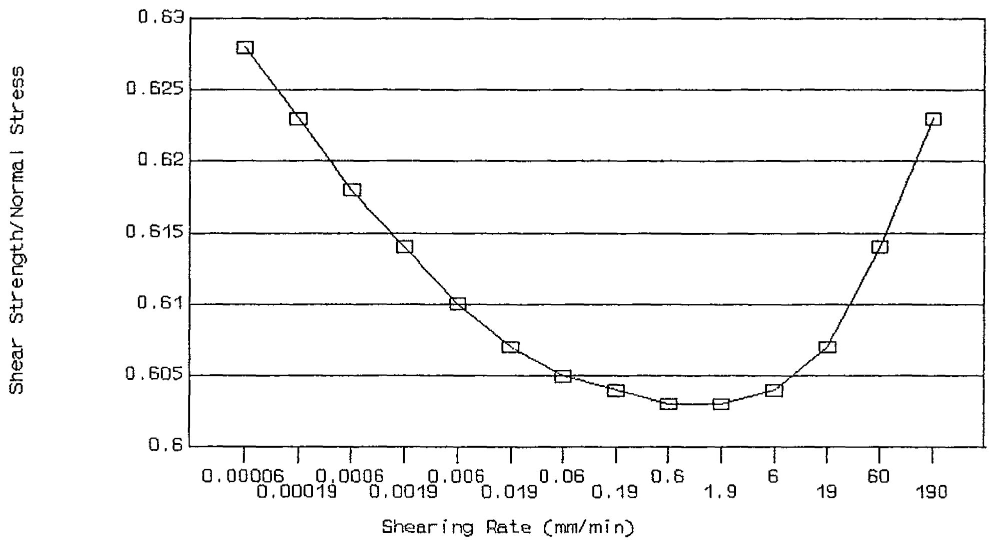



Figure 5.2 Rate Effect Study on Granite

$$\tau = \sigma \{\mu_* + a \ln (v/v_*) + \Sigma_i b_i\Omega_i\} \qquad \text{Eqn 5.4}$$

where $d\Omega_i/dt$ = $-(v/L_i)\{\Omega_i + \ln (v/v_*)\}$ ; for i = 1,2

$\mu_*$ = coefficient of friction at standard shearing rate

v = shearing rate

$v_*$ = standard shearing rate

$a - \Sigma_i b$ = steady state viscosity

$L_i$ = characteristic length

The equation shows the shear strength to be a summation of three distinct components, all of which are proportional to the normal stress. The first term is the familiar coefficient of friction. The second term introduces instantaneous changes in the shear strength due to rate effects, and provides for changes in strength proportional to the logarithm of the shearing rate. This term is similar to that provided in Equation 5.3 to descibe the rate effects of steel. The third component is a transient term that has been derived from experimental data to provide for the observed short-term changes in strength immediately following a sudden change in shearing rate.

The importance of the observed transient effects is difficult to ascertain. While they may be phenomena natural to shearing rock, they could result from imperfections in the gearing system of the ring shear apparatus. The characteristic length over which these effects apply is only 0.007mm. It is unlikely that a gearing system is capable of producing an instantaneous change in the shearing rate, particularly at very slow rates of shearing. Furthermore, any change in strength will result in a small elastic response from the gear drive which will also cause a temporary distortion in the shearing rate. It is notable that these transient factors were most significant at slow rates and it is therefore possible that these are machine, not material, effects.

Significantly, the steady state rate effects observed by Blanpied et al indicate behaviour very similar to those determined for Temuka Clay in this research project. In both cases, the reverse rate effect occurs at rates less than 1 mm/min and the positive rate effect occurs at greater rates.

Others researching the mechanics of faulting, have conducted laboratory tests on granite at different shearing rates. While none have tested samples over such an extensive range of shearing rates, their work is worth mentioning briefly.

Scholz and Engelder (1976a) report a series of tests in which samples of Westerly Granite were sheared against sapphire at rates between 0.006 and 0.6 mm/min. The tests showed a reverse rate effect over the entire range, consistent with that reported by Blanpied et al. However, less certainty is associated with these results, as they were from a direct shear, with the direction of shear being regularly reversed, and displacements limited to less than 10 mm.

Stesky (1978) reports frictional sliding tests on Westerly Granite, conducted at 250 MPa, and at temperatures ranging from 300° to 700°. He observed a conventional rate effect. Solberg and Byerlee (1984) report similar results. In testing samples of crushed, vacuum dried Westerly Granite over at rates between 0.0004 and 0.4 mm/min, the strength was observed to increase with increasing shearing rate.

In the tests by Stesky and Solberg & Byerlee, in which conventional rate effects were observed, the samples of granite were dry. In the previously mentioned tests, in which reverse rate effects were observed, the granite was sheared in the presence of water, which suggests that the presence of water has a significant effect on the rate sensitivity of rock. This important observation will be considered alongside the results of rate effects studies on other rocks.

5.2.2 Dolomite

The frictional behaviour of dolomite marble was extensively studied by Weeks and Tullis (1984, 1985) at Brown University. Tests were conducted in a rotary shear apparatus at a normal stress of 75 MPa, and at room temperature. The dolomite was tested with the shear zone material at four different moisture contents, and the shearing rate was varied from 0.0007 to 180 mm/min. Figure 5.3 shows the results.

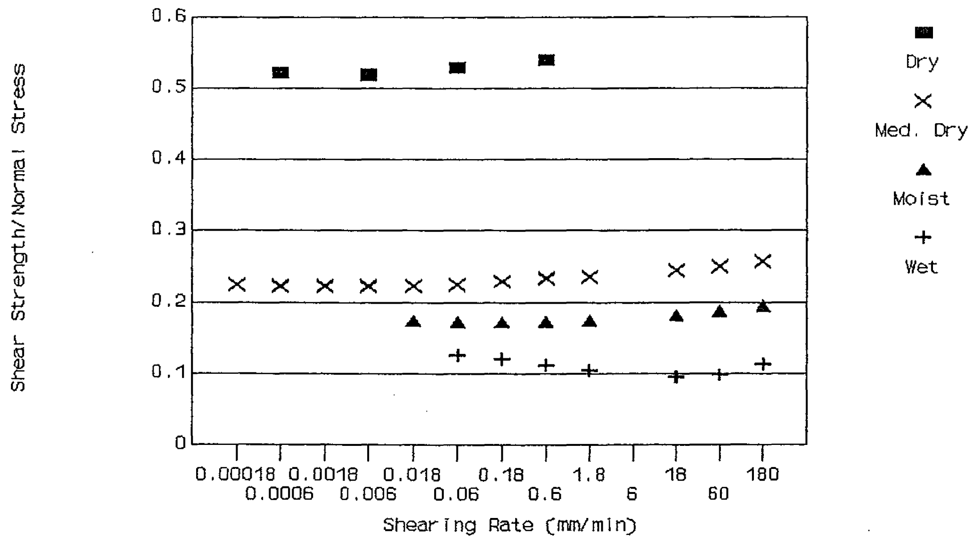



Figure 5.3 Rate Effect Study on Dolomite

Three important conclusions were drawn from the experiments. Firstly, it was observed that the strength measurements at each velocity were not affected by the previous shearing rate. Results obtained during a series of tests at different rates were found to be independent of whether the shearing rate was increasing or decreasing. This suggests that a unique shearing rate - strength function exists.

Secondly, the dolomite strength was very sensitive to the moisture content of the shearing layer. The shear strength varied from 0.08 of the vertical load (for the wettest sample) to 0.54 (for the driest sample).

Thirdly, the rate effect behaviour was significantly affected by moisture content, as illustrated in Figure 5.3. At the slowest rate, all samples exhibited the reverse rate effect and at the fastest rate, the positive rate effect. At an intermediate speed, the strength passes through a minimum. It is interesting to note that the minimum point from each series moves to higher velocities with increasing moisture content.

### 5.2.3 Halite

Shimamoto (1986) reports on a series of tests conducted on thin layers of natural and synthetic halite in a triaxial apparatus. The tests were part of research by Shimamoto & Logan (1986) into earthquake mechanisms; in particular, the stick-slip behaviour of geologically important silicate faults.

The samples of halite were tested at room temperature over a range of normal stresses from 10 to 250 MPa. The rate of shearing was varied from 0.00018 mm/min to 18 mm/min.

At low stresses, the reverse rate effect was observed up to rates of approximately 0.2 mm/min. Above this rate a small positive rate effect was observed. With increasing stress, the switch from reverse to positive rate effects occurred at slower shearing rates and the positive rate effect became far more pronounced. In some cases, at very high confining stresses, and at very slow rates, the conventional rate effect was observed for the first few readings.

### 5.2.4 Quartz

Teufel and Logan (1978) conducted a series of triaxial tests on a quartz sandstone from Tennessee at shearing rates ranging from 0.0006 to 6 mm/min. The tests were conducted at room temperature, and under a confining pressure of 50 MPa. Stick-slip behaviour was observed for most of the slow shearing rate tests. The deviatoric stress was observed to decrease with increasing shearing rate over the complete range. This reverse rate effect is consistent with the observations for other rocks.

### 5.2.5 Discussion of Rock Rate Effects

Important conclusions can be drawn from the rate effect studies on shearing rock, summarized here, and in Table 5.1. Firstly, the type of rate effect observed varied, depending on the presence of moisture. With water present, all the rocks showed reverse rate effects at slow rates of shearing and positive rate effects at fast rates. In cases where moisture was absent (due to pre-drying or because tests were conducted at very high temperatures), the tests showed conventional and positive rate effects. It is interesting to note that the observed rate effects did not significantly vary between the rock types tested.

The observed decrease in strength with increasing shearing rate for rock at slow rates is inconsistent with the behaviour of other materials. Metals, plastics and fluids all exhibit conventional rate effects. This suggests that there is some unique mechanism occurring at the rock interface that makes the rate effect behaviour differ from that observed of most substances. Clues to the nature of this mechanism are provided in other tests of shearing along rock interfaces.

Table 5.1 Summary of Observed Rate Effects for Rocks

| AUTHOR(S) | TESTING APPARATUS | ROCK TYPE | NOTES | OBSERVED RATE EFFECT |
|---|---|---|---|---|
| Blanpied et al (1987) | Ring Shear | Westerly Granite | Moisture Added | Reverse & Positive |
| Scholz and Engelder (1976a,b) | Indentation Creep | Westerly Granite | Room Tem & Moist. Content | Reverse |
| Stesky (1978) | Triaxial | Westerly Granite | High Temp. | Conventional |
| Solberg and Byerlee (1984) | Triaxial | Westerly Granite | Vacuum Dried | Conventional |
| Weeks and Tullis (1984, 85) | Ring Shear | Dolomite | Tests at Various M.C. | Reverse |
| Shimamoto (1986) | Triaxial | Halite | Tests at Various Stresses | Reverse & Positive |
| Teufel and Logan(1978) | Triaxial | Quartz | Temp. & Contact Studied | Reverse |

Dieterich (1978) reports an interesting study on the time dependent friction of rocks and the mechanics of stick-slip motion. In a series of tests on granite, greywacke, quartzite and sandstone, Dieterich studied the coefficient of friction in relation to the time of stationary contact. The conclusion was that the coefficient of friction for all rock types tested increased with increased stationary time, and that the increase was approximately proportional to the logarithm of the time of stationary contact. This increase was only significant when the stationary time exceeded one second. This is conceptionally consistent with the tests already discussed, which showed increases in strength as the shearing rate is slowed.

These observations suggest a time controlled strengthening of grain to grain contacts. The mechanism responsible for this time dependency could be one of two possibilities. The contact area of the rock may increase as a consequence of time-dependent reductions in the surface hardness due to creep. Alternatively, the actual area may remain constant, but the strength of the contact points may increase due to a time-dependent breakdown of the surface film that interferes with adhesion.

Experimental work by Scholz and Engelder (1976a,b) indicates that the contact area does increase as a function of time. When samples of quartz and olivine were subjected to a constant load, the area of indentation was found to increase proportionally to the logarithm of the loading duration. The increase in strength resulting from increased indentation is likely to be greatest when different minerals are sheared against each other. This is due to asperities from the harder rock mineral ploughing into the softer mineral.

This explanation suggests that the increased strength at slower rates represents an increase in the contact area due to time-dependent creep of the surface indentations. This conclusion is confirmed by Teufel and Logan (1978) who used dye to determine the contact area for triaxial tests on Tennessee Sandstone. Their results showed a marked decrease in the area of contact as shearing rate was increased from 0.0006 to 6 mm/min.

The presence of water has also been found to have a significant effect on the reduction in surface hardness by creep. In tests at room temperature and moisture content, Westbrook and Jorgensen (cited in Dieterich, 1978) found a significant drop off in surface hardness for a range of rock minerals including quartz and calcite. When the same tests were conducted in a water free environment (argon), no time dependent surface softening was observed.

Together these results provide a consistent explanation of the observed behaviour of the rate effects of shearing rock.

At slow shearing rates and in the presence of small amounts of moisture, the slower the shearing rate, the greater the indentation creep causing a small increase in the residual shear strength. Without moisture being present, indentation creep is insignificant and the rock behaves similarly to other materials, showing an increase in the strength with increasing shearing rate. At fast rates of shear, creep indentation does not have sufficient time to be significant and rock displays positive rate effects.

## 5.3 ARTIFICIAL GRANULAR MATERIALS

The study of shear in granular materials is complicated by the fact that most naturally occurring materials include a wide range of particles of different shapes and sizes. Some researchers have simplified the problem by undertaking laboratory investigations into the shear behaviour of artificial materials with uniform particle size and shape.

Bagnold (1954) undertook an experimental study into the behaviour of a gravity-free dispersion of large solid spheres in a Newtonian fluid under shear. The study focused on bridging the gap between pure fluid mechanics and geomechanics, by considering intermediate solid - fluid mixtures. Spheres 1.3mm in diameter, and of the same density as water, were mixed in various concentrations with water in an annular space between two concentric drums. The outer drum was rotated and the torque on the inner drum measured. Positive rate effects were observed in all the tests. In varying the rate of shear over the 10.8 mm thick mixture from 4.8m/min to 167m/min, the shear strength of the highest concentration mixture increased from 0.4 to 6.4 Pa. The increase in the strength was found to be approximately proportional to the square of the rate of shearing.

It is important to note that these tests vary from those investigating the behaviour of soils in that no normal stress is applied across the direction of shearing. The results are effectively a measurement of the effect of rate of shear on the cohesion component of a soil's strength. In the case of sands, the cohesion is often a negligibly small component of total strength.

In order to analyse the flow of granular materials in industrial applications, Novasad (1964) conducted a series of tests in the Hvorslev ring shear apparatus, at low normal stresses, and fast shearing rates. The two materials tested were glass spheres and crystalline sugar, where the average particle sizes were 2.5mm and 0.5mm, respectively. The samples were subjected to normal stresses ranging from 0.5 kPa to 2.3 kPa, and the shearing rate was varied from 280 mm/min to 28 m/min. Novasad concluded that for both the artificial materials, the shearing rate had a negligible effect on the measured shear stress.

Savage (1982) tested three types of materials: glass beads, polystyrene spheres and crushed walnut shells, using an annular shear apparatus. The dry samples were sheared at rates between 5 and 80 mm/min, over which range the shear strength did not vary significantly.

Similar work was done by Hungr and Morgenstern (1984) to study the behaviour of polystyrene beads at shear rates from 60 to 60 000 mm/min. The tests were conducted at normal stresses of up to 70 kPa. Greater stresses were not attempted as very strong stick slip behaviour threatened to damage the apparatus. Identical shear strengths were measured at rates from 60 to 9600 mm/min. At the fastest rate of 60,000 mm/min, stick-slip behaviour prevented the measurement of the residual strength.

Ring shear tests on medium density dry glass beads, at constant normal stress were undertaken by Sassa (1984), in conjunction with Japanese research into debris flows. The tests showed that over a range of shearing rates between 0.00008 and 15 mm/min, the shear strength remained constant.

Tika (1989) reports a series of tests on mixtures of sands and viscous fluids. The tests were conducted in the Bishop Ring Shear Apparatus at rates from 0.1 to 6000 mm/min and over a range of normal stresses from 40 to 230 kPa. A fine sand - glycerol mixture exhibited negative rate effects at all normal stresses. The medium sand - silicon oil mixture showed minimal rate effects except at the lowest normal stress, where it showed positive rate effects.

The rate effect studies of artificial granular materials are summarised in Table 5.2 on the following page. The results indicate that, generally, these materials have negligable rate effects at both slow and fast rates. The sand - glycerol mixture tested by Tika (1989) is the exception. The very small positive rate effects observed in some tests, at low normal stresses and at very fast rates, are not unexpected. These can be attributed to the viscous properties of the component fluid.

Table 5.2 Summary of Observed Rate Effects For Artificial Granular Materials

| AUTHOR(S) | TESTING APPARATUS | SOIL DESCRIPTION | NORMAL STRESS | SHEARING RATE RANGE (mm/min) | STRENGTH ACCURACY | OBSERVED RATE EFFECTS |
|---|---|---|---|---|---|---|
| Bagnold (1954) | Concentric Drums | 1.33mm Beads Water,Alcohol | 0 kPa | 4,800 - 167,000 | - | Positive |
| Novasad (1964) | Ring Shear (Hvorslev) | 2.5mm Glass Spheres | 0.5-2.3 kPa | 280 - 28,000 | 4 % | None Observed |
| | | .5mm Crystals of Sugar | 0.5-2.3 kPa | 280 - 28,000 | 4 % | None Observed |
| Savage (1982) | Concentric Drums | Glass Beads | - | 5 - 80 | - | None Observed |
| | | Polystyrene Spheres | - | 5 - 80 | - | None Observed |
| | | Crush. Walnut Shells | - | 5 - 80 | - | None Observed |
| Hungr and Morgenstern (1984) | Ring Shear (Hvorslev) | Polstyrene Beads | 20-200 kPa | 300 - 60,000 | 4 % | None Observed |
| Sassa (1984) | Ring Shear (Sassa) | 1.0 mm Glass Beads | 8-20 kPa | 0.00008 - 15 | - | None Observed |
| Tika (1989) | Ring Shear (Bishop) | Sand-Glycerol Mixture | 40-400 kPa | 0.1 - 6000 | 2 % | Negative |
| | | Sand-Silicon Oil Mixture | 40-400 kPa | 0.1 - 6000 | 2 % | Positive |

## 5.4 COHESIONLESS SOILS

Studies of rate-dependence in the strength of coarse-grained soils have been conducted by a number of researchers. The apparatus and rates of shearing used vary considerably, but together they provide a database from which general conclusions can be drawn that enhance the understanding of rate effects.

In work undertaken by Novasad (1964) to study the flow of granular materials, 0.2 mm and 1.0 mm uniform sands were subjected to fast shearing rates in the Hvorslev ring shear apparatus, with low normal stresses ranging from 0.5 to 2.3 kPa. Novasad concluded that over the range of shearing rates, from 280 to 28,000 mm/min, changes in strength were negligible.

Atakol and Larew (1970) report very fast shear tests on dry Ottawa sand in a simple shear apparatus. The tests were conducted at normal stresses ranging from 2.9 kPa to 574 kPa, and the rate of shear was varied from 18 mm/min to 610 m/min. The very fast shearing rates were generated by a projectile fired from an explosive charge actuator.

A complex testing methodology was developed to eliminate inertia effects of the apparatus, and to isolate the dynamic strength characteristics of sand at fast shearing rates. The results indicated that the shearing resistance did not change substantially for rates up to 36 m/min but, beyond this rate increased substantially up to a strength 50% in excess of the static strength. The strength stabilised at rates in excess of 200m/min. The increase in strength was most pronounced at high normal stresses.

The effect of the penetration rate in cone penetration tests, using dry sand, was studied by Dayal and Allen (1975). Laboratory tests on both dense and loose sand were conducted at shearing rates ranging from 78 mm/min to 49 m/min. The results indicated that there was no appreciable difference in either the cone or friction sleeve resistance as a consequence of the variations in the penetration rate.

A fine sand from the North Sea seabed was tested in a special apparatus by Heerema (1979). The apparatus enabled a saturated and confined sample of sand to be sheared to displacements of up to 12.5 mm. Larger displacements were generated by continuously forwarding and reversing the apparatus. A horizontal load was applied to a steel plate to provide confining stresses from 80 to 240 kPa. Variations in the shearing rate from 42 mm/min to 36 m/min made no appreciable difference to the measured strength, and Heerema concluded that the strength was independent of the velocity of shearing.

Hungr and Morgenstern (1984) conducted fast rate ring shear tests on a number of materials in search of an explanation for the high mobility of sturzstroms. The ring shear apparatus used was similar to the Hvorslev design and enabled rates of shearing of 300 mm/min, 9.6 m/min and 60 m/min at normal stresses up to 200 kPa. Tests were carried out on wet and dry coarse sand and sand - rock flour mixtures. The sand and the rock flour were both quartz and had an average particle size of 1.7mm and 0.044mm respectively. No rate effects were observed for any of the tests to within an accuracy of 4%.

Artificial sand, obtained by mixing medium and coarse sand, was tested in the Bishop ring shear apparatus by Lemos (1986). The saturated samples were tested at normal stress of 100 kPa, at shearing rates between 0.0038 mm/min and 133 mm/min. No appreciable rate effects were detected.

Tika (1989) also investigated the rate effects on the shearing of sand in the Bishop ring shear apparatus. The tests were conducted on both medium and fine silica sand at three different normal stresses of 115 kPa, 220 kPa and 445 kPa, and at shearing rates ranging from 0.1 to 8000 mm/min. The results indicated that any change in strength as a consequence of variations in the shearing rate were very small and likely to be less than 5% over the entire range of shearing rates. The readings did indicate a slight positive rate effect at rates in excess of 10 mm/min, but the data is too scattered relative to the small effect for this to be concluded with confidence.

Tika (1989) also reports similar tests using a sandstone from the Kalabagh Dam project. The clay fraction was predominantly montmorillonite in mineral content, and the sand fraction exceeded 75%. At a normal stress of 400 kPa the residual shear strength increased by approximately 2% between shearing rates of 100 and 8000 mm/min. At other rates at 400 kPa, and at all rates at 200 kPa, the scatter of data prohibited any conclusions from being drawn about rate effects other than that they would be very small.

A summary of these studies on the rate effects on the residual shear strength of sands is provided in Table 5.3 on the following page. These results indicate that variations in the rate of shearing have only a minimal impact on the residual shear strength. It is significant that none of the results showed negative or reverse rate effects. The only tests that gave positive rate effects were those conducted at exceptionally high speeds ( >100,000 mm/min) or at a very high level of accuracy. It is concluded that sands exhibit only very weak, if any, rate effects.

Table 5.3 Summary of Observed Rate Effects for Cohesionless Soils

| AUTHOR(S) | TESTING APPARATUS | SOIL DESCRIPTION | NORMAL STRESS | SHEARING RATE RANGE (mm/min) | STRENGTH ACCURACY | OBSERVED RATE EFFECTS |
|---|---|---|---|---|---|---|
| Novasad (1964) | Ring Shear (Hvorslev) | 1.0mm Coarse Sand | 0.5-2.3 kPa | 280 - 28,000 | 4 % | None Observed |
| | | 0.2mm Fine Sand | 0.5-2.3 kPa | 280 - 28,000 | 4 % | None Observed |
| Atakol and Larew(1970) | Simple Shear | Dry Ottawa Sand | 2.9-574 kPa | 18 - 610,000 | 10 % | Positive |
| Dayal and Allen(1975) | Stand. Pen. Cone | Dense & Loose Sand | - | 78 - 49,000 | 5 % | None Observed |
| Heerema (1979) | Model Pile | Fine Sand | 80-240 kPa | 42 - 36,000 | 5 % | None Observed |
| Hungr and Morgenstern (1984) | Ring Shear (Hvorslev) | Wet Coarse Sand | 20-200 kPa | 300 - 60,000 | 4 % | None Observed |
| | | Dry Coarse Sand | 20-200 kPa | 300 - 60,000 | 4 % | None Observed |
| | | Sand - Rock Flour Mixture | 20-200 kPa | 300 - 60,000 | 4 % | None Observed |
| Lemos(1986) | Ring Shear (Bishop) | Art. Sand Mixture | 100 kPa | 0.0038 - 133 | 2 % | None Observed |
| Tika (1989) | Ring Shear (Bishop) | Fine Sand | 115-445 kPa | 0.1 - 8000 | 2 % | Slight Positive |
| | | Medium Sand | 115-445 kPa | 0.1 - 8000 | 2 % | None Observed |
| | | Sandstone | 400 kPa | 100 - 8000 | 2 % | Slight Positive |

## 5.5 COHESIVE SOILS

The study of rate effects is more complex in cohesive soils. This complexity derives from the difference between the post peak strength behaviour of fine and coarse soils as illustrated in Figure 5.4.

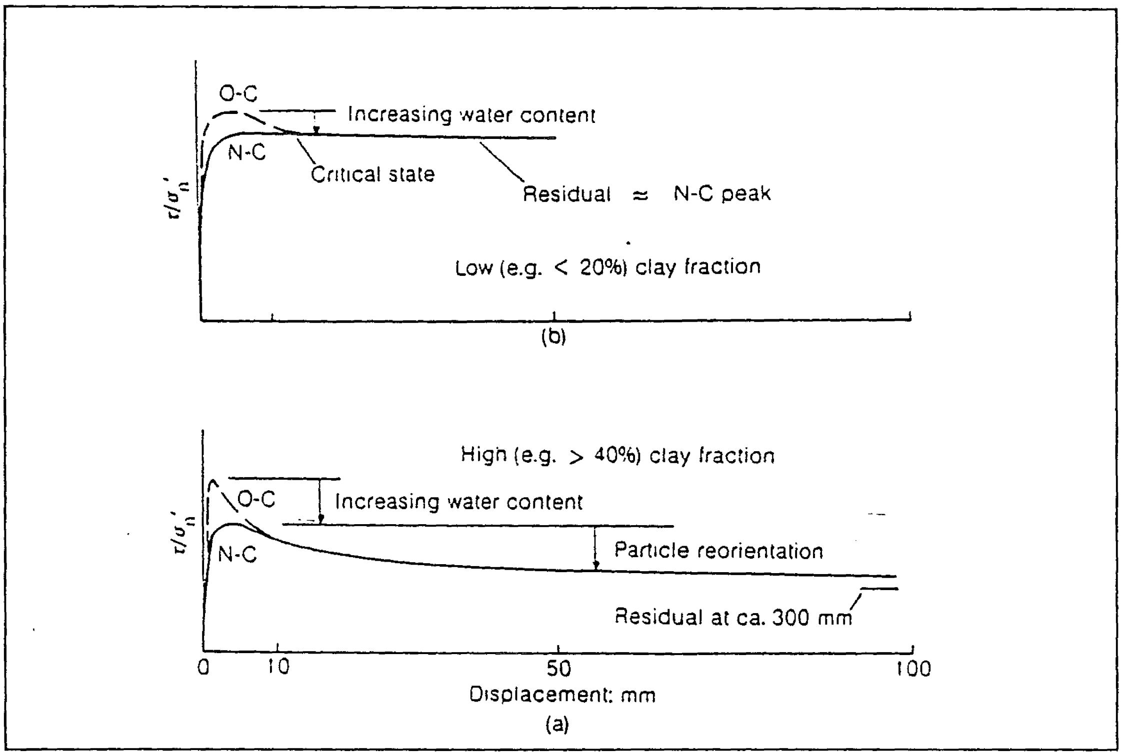



Figure 5.4 Stress-Displacement Curves (Skempton 1985)
(a) Fine Grained Soil (b) Coarse Grained Soil

When a coarse grained soil is sheared, the post-peak drop in strength is due to dilatation and the increasing water content of the soil as it approaches its critical state. The shear displacement required to reach true residual conditions is often only a few millimetres (or particle diameters).

For fine grained soils with a high clay content, two mechanisms are simultaneously occurring which result in a post-peak drop in strength. As for the coarse grained material, the shearing zone dilates and increases in moisture content resulting in a drop in strength. However, due to the low permeability of fine grained materials, the time required for this to occur can be prolonged. A further mechanism is also at work. With increasing shear displacement, the platy clay particles tend to align themselves with the direction of shearing, thus further decreasing the residual shear strength. For some fine grained soils, the displacement required to complete the process of particle re-orientation can be several metres.

Studies of rate effects in fine grained soils in the ring shear apparatus are further complicated by pore pressure effects. As analysed in Chapter 3, soil losses from the ring shear apparatus may generate significant pore pressures when testing fine soils, and this may distort measurements of the residual shear strength.

In the following sub-sections, the rate effects for fine soils observed by other researchers will be summarised and compared. The results will be classified into three groups. The first set contains results of tests not conducted in the ring shear apparatus, and includes all of the early rate effect studies. The second group includes all the work conducted at Imperial College using the Ring Shear Apparatus developed by Bishop in 1971, and the final group includes those ring shear studies not conducted at Imperial College.

5.5.1 Non Ring Shear Studies

The significance of rate effects on the strength measurements of clays was recognised in early studies by Casagrande and Wilson (1951). A series of tests at different rates of loading were conducted on a range of clays and shales in an unconfined compression apparatus. The rates of loading were sufficiently slow so that all of the tests were conducted in the equivalent of the slow linear range. Five samples showed conventional rate effects and four reverse rate effects. The apparatus did not enable sufficient displacement to ensure that true residual conditions had developed but the results, nevertheless, indicate soil responses to variations in the rate of loading.

As part of a study of the stability of landslides in South East California, Gould (1960) reported a series of triaxial shear tests at different rates on a bituminous clay. Both weathered and unweathered samples obtained from the failure plane of a landslide were tested over a wide range of slow loading rates. The strength of the soil was found to increase with increasing shearing rate: a conventional rate effect. This result is compromised in the same way as the results reported by Casagrande, in that the triaxial apparatus limits the shearing to only small displacements.

A series of direct shear tests on Brown London Clay to determine the appropriate shearing rate for laboratory testing is reported in Petley (1966). The pre-cut samples were sheared at rates ranging from 0.00004 to 0.1 mm/min and the measured strength increased by 4% over the entire range: indicating a conventional rate effect. He concludes that the difference between the field and laboratory rates of shearing would not have a significant effect on the shear strength. Confidence in this result must be tempered by the fact that the direction of shearing had to be regularly reversed in the direct shear apparatus.

Kenney (1967) reports a series of direct shear tests on pre-cut samples of various minerals, including kaolinite, muscovite and montmorillonite. At a normal stress of 98 kPa, and at shearing rates between 0.000001 to 0.1 mm/min, small conventional rate effects were observed for all the samples.

A series of multiple reversal direct shear box tests on a Bombay clay silt are described in Ramiah et al (1970). The normal stress was varied from 50 to 164 kPa and the shearing rate from 0.02 to 10 mm/min. The results indicate a small reverse effect at slow rates and a small positive effect at a shear rate over 1 mm/min. The combination of reverse and positive rate effects is similar to that observed in this study on Temuka Clay.

Cullen & Donald (1971) report direct shear tests at Monash University, Melbourne, on Silurian Clay. Normal stress was 207 kPa, shearing rates ranged from 0.0167 to 0.25 mm/min.

With multiple reversals of the apparatus, the results of the 8 tests show conventional rate effects.

Rate effects were studied by Dayal and Allen (1975), who drove a standard penetrometer into laboratory-prepared samples of pottery clay. The cone pressure and sleeve friction was measured for a range of different penetration rates from 80 to 49,000 mm/min. Over the entire range of fast rates, both the cone pressure and sleeve friction increased with increasing shearing rates. This is consistent with a positive rate effect.

A model pile apparatus was used by Heerema (1979) to study the rate effects on North Sea Clay at medium to fast rates. The apparatus sheared a horizontally loaded steel plate against an undisturbed soil sample. The vertical displacements were limited to 12.5mm, and so the direction of movement was oscillated at quite high frequencies for the fast rate tests. In all the tests the shear strength was observed to increase with increasing shearing rate and this effect was most pronounced at fast rates.

Rate effect studies for tests not conducted in the ring shear apparatus are summarized in Table 5.4. At fast rates all of the results indicate positive rate effects. At slow rates both reverse and conventional rate effects are observed. The conventional rate effect seems to coincide with soils of greater clay content and plasticity, while the reverse rate effect is observed for soils with greater silt fraction. This hypothesis will be reassessed when the reported ring shear tests have been considered.

Table 5.4 Summary of Rate Effect Studies on Cohesive Soils in Apparatus other than the Ring Shear

| AUTHOR(S) | TESTING APPARATUS | SOIL DESCRIPTION | RATE EFFECTS | |
|---|---|---|---|---|
| | | | Slow Rates | Fast Rates |
| Casagrande and Wilson (1951) | Unconfined Compression | Mexico City Clay | Conventional | - |
| | | Vicksburg Silty Clay | Reverse | - |
| Gould (1960) | Triaxial | Modelo Clay | Conventional | - |
| Petley (1966) | Direct Shear | Brown London Clay | Conventional | - |
| Kenney (1967) | Direct Shear | Kaolinite | Conventional | - |
| | | Muscovite | Conventional | - |
| | | Montmorillon. | Conventional | - |
| Ramiah et al (1970) | Direct Shear | Bombay Clay Silt | Reverse | Positive |
| Cullen & Donald (1971) | Direct Shear | Silurian Clay | Conventional | - |
| Dayal and Allen (1975) | Standard Penetration | Pottery Clay | - | Positive |
| Heerema (1979) | Model Pile | North Sea Clay | - | Positive |

### 5.5.2 Imperial College Studies

The residual strength of fine grained soils has been the subject of an on-going research programme at Imperial College that has spanned more than two decades. This research was built on the pioneering work in the ring shear apparatus by Hvorslev (1939) who concluded that the ring shear apparatus was the most appropriate test to determine the residual strength of fine grained soils.

In Bishop et al (1971), a new design of the ring shear apparatus is reported. Its design and its modifications of the Hvorslev machine are summarised in Chapter 2. Interest in the effects of shearing rate on residual strength were initially motivated by the desire to prevent differences between the laboratory and field shearing rates from distorting interpretations of the residual shear strength. Later studies focused on the implications of rate effects for the design and installation of piles in cohesive soils. More recently, studies have centred on the residual shear strength at fast rates, because of the significant impact rate effects may have on slide behaviour during earthquake events.

As part of the initial investigation into the behaviour of the new apparatus, Bishop et al (1971), documented ring shear tests on Blue London Clay, conducted at two different slow shearing rates. The change in strength indicated a small conventional rate effect. They concluded that variations in the laboratory shearing rate did not have a significant effect on the measured residual shear strength.

A more extensive rate effect study on a number of soils is reported by Lupini et al (1981). The first series of tests was for a residual soil from the Fijian Islands. The sample was sheared at a normal stress of 177 kPa, at shearing rates ranging from 0.0023 to 177 mm/min. At slow rates conventional rate effects were observed. At fast rates the strength was observed to rise substantially above the slow residual value and then quickly decline to very low strengths. This negative rate effect has major implications for slope stability problems, as it suggests that the soil strength may collapse once a high rate of shearing is attained.

In the same report, Lupini et al describe ring shear tests on Lower Cromer Till at a normal stress of 221 kPa. The shearing rate was varied from 0.0001 to 133 mm/min. At low shearing rates, the strength was observed to decrease with increasing shearing rate. At very high rates of shearing, the shear strength dropped to less than the slow residual value, indicating a negative rate effect very similar to that for the residual Fiji soil.

The same researchers reported results for Blue London Clay. At slow rates of shear and at a normal stress of 352 kPa, a slight reverse rate effect was observed; while at a normal stress of 220 kPa conventional and positive rate effects were observed.

Their tests on Kaolin, at a normal stress of 519 kPa, over a range of shearing rates from 0.0001 to 130 mm/min indicated that the soil exhibited both conventional and positive rate effects.

Martins (1983) undertook two ring shear tests in the Bishop apparatus as part of a study into the shaft resistance of axially loaded piles in clays. Both weathered and unweathered Panama Tuff were tested at normal stresses of 100 and 300 kPa. The results over a range of shearing rates from 0.0006 to 100 mm/min indicated weak conventional rate effects and strong positive rate effects.

A specific study into the rate effects on residual strength is reported in Lemos (1986). The study involved shearing a wide range of soils at various shearing rates and also included tests on the strength of soil/solid interfaces.

The first soils tested by Lemos were from two landslides in the South of Italy which were reactivated by an earthquake in 1980. Two samples from the Senerchia slide were tested at normal stresses ranging from 200 to 900 kPa and at shearing rates between 0.00125 and 133 mm/min. The results indicated reverse and positive rate effects. A sample from the Boniventre slide, tested over a similar range of normal stresses and shearing rates exhibited conventional and positive rate effects.

The same report describes tests on samples of weathered and unweathered Cowden Till at normal stresses ranging from 50 to 200 kPa. At slow rates between 0.0006 and 0.1 mm/min, both the weathered and unweathered samples showed negligible rate effects. The weathered sample was tested at fast rates of up to 400 mm/min, and showed positive rate effects.

Lemos tested a glacial till from the Magnus platform in the North Sea at a normal stress of 500 kPa. Over a shearing range from 0.017 to 131 mm/min, the soil showed weak conventional and strong positive rate effects.

He also reported an extensive series of fifteen ring shear tests on seven soils from the Kalabagh dam project in Pakistan. Because the dam was to be constructed in an area of high seismicity, it was considered important to understand the behaviour of the soils at high shearing rates. A range of normal stresses from 180 to 530 kPa were used on a number of siltstones and claystones, and the shearing rate was varied from 0.006 to 6200 mm/min. The testing programme was designed to isolate rate effects at fast rates. Consequently, there is insufficient data to draw firm conclusions about rate effects at slow rates except that the effects were small over the range of rates tested. At fast rates of shearing, the samples showed a very wide diversity of behaviour, ranging from strong negative to strong positive rate effects. Identical samples of clayey silt showed opposite behaviour from one test to the next. Interpretation of the results is complicated by the fact that the soils were tested at different normal stresses, consolidation ratios and moisture contents, and no simple explanation is provided for the diverse behaviour.

Tika (1989) continued Lemos' study into the rate effects on the residual strength of cohesive soils at Imperial College. She conducted a further eleven tests on soil samples from the Kalabagh dam project.

The first series of tests was sheared at mid-depth in the Bishop apparatus in a procedure similar to that used by other researchers at Imperial College. The claystone sample was tested at four normal stresses ranging from 70 to 530 kPa, over a range of shearing rates from 0.01 to 2600 mm/min. At the fastest rates, at all the normal loads, the sample showed strong negative rate effects, but at an intermediate rate of 360 mm/min, a small positive rate effect was observed.

A clayey siltstone from the same site was tested at normal stresses from 265 to 530 kPa. In five tests at shearing rates ranging from 0.0053 to 5400 mm/min, negative rate effects were observed at every fast rate.

To further investigate these phenomena, the natural clayey siltstone was artificially modified by reducing the clay fraction in a sedimentation test. The modified sample was sheared over a wide range of rates at a normal stress of 240 kPa. At slow rates negligible rate effects were observed, while at fast rates the sample showed strong negative rate effects.

An additional four samples were sheared against a roughened surface of glass or stainless steel and showed rate effect behaviour very similar to that observed in the conventional Bishop tests, where the sample was sheared at mid-depth. It was noted, on dismantling the samples after testing, that the shearing zone had formed adjacent to, but not at, the roughened surface.

Tika carried out a normal ring shear test on Lower Cromer Till, the same soil tested by Lupini in 1981, at normal stresses ranging from 50 to 572 kPa. Lupini had reported reverse rate effects at slow shearing rates and Tika's results confirmed this. At fast shearing rates of up to 1200 mm/min, all the test results indicated negative rate effects.

In another series, residual soil from Fiji (tested by Lupini in 1981) was retested by Tika. Lupini had reported that at rates of 177 mm/min, the sample exhibited negative rate effects. At normal stresses of 200 kPa, these results were confirmed by Tika at rates of 1100 and 2400 mm/min.

Mauritius Silt from the Champagne hydro-electric project in France was tested at normal stresses of 220 and 450 kPa, at shearing rates from 0.01 to 2000 mm/min. At slow rates the sample showed a slight conventional rate effect, while at fast rates strong negative rate effects were observed.

Two samples of silt from the exposed faces of the huge Mt Ontake landslide were also tested. At normal stresses from 130 to 480 kPa and at shearing rates ranging from 0.01 to 6000 mm/min, the samples exhibited negligible rate effects.

Tika created two artificial soils from silt/kaolin and sand/kaolin mixtures, to further investigate the rate effects for intermediate soils. The clayey silt was studied at normal stresses of 60 and 480 kPa and at shearing rates from 0.01 to 2500 mm/min. It exhibited significant positive rate effects. The sandy clay was tested over the same range of shearing rates at normal stresses of 200 and 450 kPa, and it showed slight positive rate effects.

Two tests were carried out on Blue London Clay. The overconsolidated samples were tested at 250 and 500 kPa over a range of shearing rates from 0.006 to 6200 mm/min. Both tests indicated a weak positive rate effect. This is consistent with the results for the same soil as reported by Lupini et al in 1981. All the reported Imperial College ring shear tests studying rate effects for cohesive soils are summarised in Table 5.5.

Table 5.5A Summary of Rate Effect Studies on Cohesive Soils at Imperial College in Bishop Apparatus

| OPERATOR | SOIL DESCRIPTION | %CLAY FRACTION | NORMAL STRESS | SHEARING RATES mm/min | RATE EFFECTS | |
|---|---|---|---|---|---|---|
| | | | | | Slow Rates | Fast Rates |
| Bishop et al (1971) | Blue London Clay | 57 % | 43 - 299 kPa | 0.0038 - 0.0076 | Conventional | - |
| Lupini et al (1981) | Fiji Residual Soil | 40 % | 176 kPa | 0.0023 - 177 | Conventional | Negative |
| | Lower Cromer Till | 23 % | 221 kPa | 0.0001 - 133 | Reverse | Negative |
| | Blue London Clay | 48 % | 220 kPa | 0.001 - 177 | Conventional | Negative |
| | Kaolin | 53 % | 519 kPa | 0.0001 - 130 | Conventional | Positive |
| Martins (1983) | Panama Tuff | - | 100 - 300 kPa | 0.0006 - 100 | Conventional | Positive |
| Lemos (1986) | Senerchia Clay | 37 % | 200 - 900 kPa | 0.0012 - 133 | Reverse | Positive |
| | Bonivenre Clay | 39 % | 200 - 900 kPa | 0.003 - 400 | Conventional | Positive |
| | Cowden Till | 29 % | 50 - 200 kPa | 0.0006 - 400 | Negligible | Positive |
| | Magnus Till | 36 % | 500 kPa | 0.017 - 131 | Conventional | Positive |
| | KALABAGH DAM SOILS | | | | | |
| | Claystone A | 45 % | 210 - 500 kPa | 0.155 - 400 | - | Positive |
| | Claystone B | 40 % | 180 - 530 kPa | 0.0065 - 774 | - | Positive |
| | Claystone C | 40 % | 200 - 520 kPa | 0.01 - 400 | - | Negative |
| | Clayey Siltstone A Tests No. 3 - 8 | 20 % | 190 - 500 kPa | 0.002 - 700 | - | Negative |
| | Test No. 9 | | 480 kPa | 0.006 - 430 | - | Positive |

Table 5.5B Summary of Rate Effect Studies on Cohesive Soils at Imperial College in Bishop Apparatus (Continued)

| OPERATOR | SOIL DESCRIPTION | %CLAY FRACTION | NORMAL STRESS | SHEARING RATES mm/min | RATE EFFECTS | |
|---|---|---|---|---|---|---|
| | | | | | Slow Rates | Fast Rates |
| Lemos (1986) | KALABAGH DAM SOILS (Continued) | | | | | |
| | Clayey Siltstone B (3 Tests) | 9 % | 200 - 500 kPa | 0.006 - 6200 | - | Negative |
| | Clayey Siltstone C | 10 % | 200 - 500 kPa | 0.01 - 400 | - | Negative |
| | Siltstone | 2 % | 180 - 500 kPa | 0.01 - 400 | - | Positive |
| Tika (1989) | Claystone D | 40 % | 70 - 530 kPa | 0.01 - 2600 | - | Neg & Pos |
| | Clayey Siltstone D | 20 % | 265 - 530 kPa | 0.0053 - 5400 | - | Negative |
| | Clayey Siltstone D ( Modified ) | 13 % | 240 kPa | 0.0014 - 6000 | - | Negative |
| | OTHER SOILS | | | | | |
| | Lower Cromer Till | 21 % | 50 - 572 kPa | 0.025 - 1200 | Reverse | Negative |
| | Fiji Residual Soil | 55 % | 200 - 572 kPa | 0.016 - 2400 | - | Negative |
| | Mauritius Silt | 10 % | 220 - 450 kPa | 0.01 - 2000 | Conventional | Negative |
| | Mt. Ontake Silt | 13 % | 130 - 480 kPa | 0.01 - 6000 | Negligible | Negligible |
| | Art. Clayey Silt | 21 % | 60 - 480 kPa | 0.01 - 2500 | - | Positive |
| | Art. Sandy Clay | 34 % | 200 - 400 kPa | 0.1 - 2500 | - | Positive |
| | Blue London Clay | 80 % | 250 - 500 kPa | 0.006 - 6200 | - | Positive |

Before comparing these results with those of other research programmes, consideration must be given to the problem of sample degradation and pore pressure generation, described earlier in this thesis.

Chapter 3 demonstrates that the loss of soil from the ring shear apparatus will generate a water demand where the moisture content of the shearing soil varies from that of the original sample. Where the shearing soil increases in moisture content, this water demand will cause negative pore pressures; and where the soil decreases in moisture content, positive pore pressures will result. As discussed, the magnitude of these pore pressures will be significantly greater in the Bishop apparatus because of the greater drainage path distance (ie from the failure plane to the porous loading platens).

In section 3.3.2, the results of Lemos' test on Claystone A and Clayey Siltstone A are analysed in detail and it is concluded that the observed rate effects are due to pore pressures generated by sample degradation, rather than any structural change in the shearing process. It is not possible to complete the same analysis for all the ring shear tests reported by Imperial College. Insufficient data are available regarding soil permeabilities, degradation rates and changes in the moisture content in order to make a complete analysis. However, it is useful to qualitatively review some of the unusual results observed in cases where some of the key data is available.

It is important to note that the measurements of moisture content in the failure zone are very approximate because it is difficult to isolate a sample of the soil from the shear zone for a moisture content test. As a consequence, the following comparisons are only approximate.

Chapter 3 draws attention to Test #3 by Lemos on Kalabagh Dam Clayey Siltstone A, which showed negative rate effects. Using the data recorded for that test, this behaviour could be explained by sample degradation and the resulting pore pressures. Test #9 was conducted on the same soil but it showed the opposite behaviour: positive rate effects. While both samples had a bulk sample moisture content of 20.5%, the moisture content of the failing soil was 20 % for Test # 3, and 21.5 % for Test # 9. The difference in behaviour is, therefore, totally consistent with the concept of sample degradation and pore pressure generation.

It is also interesting to note the results for the artificial clayey soil tested by Tika. This had a very similar clay fraction to the Kalabagh clayey siltstone sample and also exhibited positive rate effects. The bulk and shearing zone moisture contents were 16.3 % and 19 % which suggests a positive water demand and a perceived positive rate effect.

This result then is also consistent with the sample degradation and pore pressure theory developed in chapter 3.

The tests conducted at Imperial College, in which the soil sample is sheared against a roughened surface, do not provide any further evidence of whether the observed rate effects are pore pressure generated. This is because the roughened surfaces, glass and stainless steel, are both non-porous. The process of sample degradation, and the development of pore pressures would occur in a way very similar to that in samples sheared at mid-depth. A test using the Bishop apparatus and a roughened porous surface might be expected to behave quite differently.

All these results strongly suggest that the fast rate tests in the Bishop ring shear apparatus may be distorted by sample degradation, and the associated water demand and pore pressures; and that the observed rate effects may have little relevance to actual soil behaviour in the field.

### 5.5.3 Other Ring Shear Studies

Ring shear studies of rate effects for cohesive soils have been undertaken by other researchers in association with fundamental soils research, and with regard to specific design problems on major projects.

The earliest study into the rate effects of ring shear testing was conducted at the Waterways Experimental Station in Vicksburg, Mississippi, by Hvorslev and Kaufman (1951). In a ring shear apparatus similar to that originally designed by Hvorslev in 1939, tests were carried out on samples of Lake Providence Clay, at a normal stress of 196 kPa . In varying the shearing rate from 0.31 to 310 mm/min, the residual shear resistance showed both reverse and positive rate effects. These results are presented in Figure 5.5.

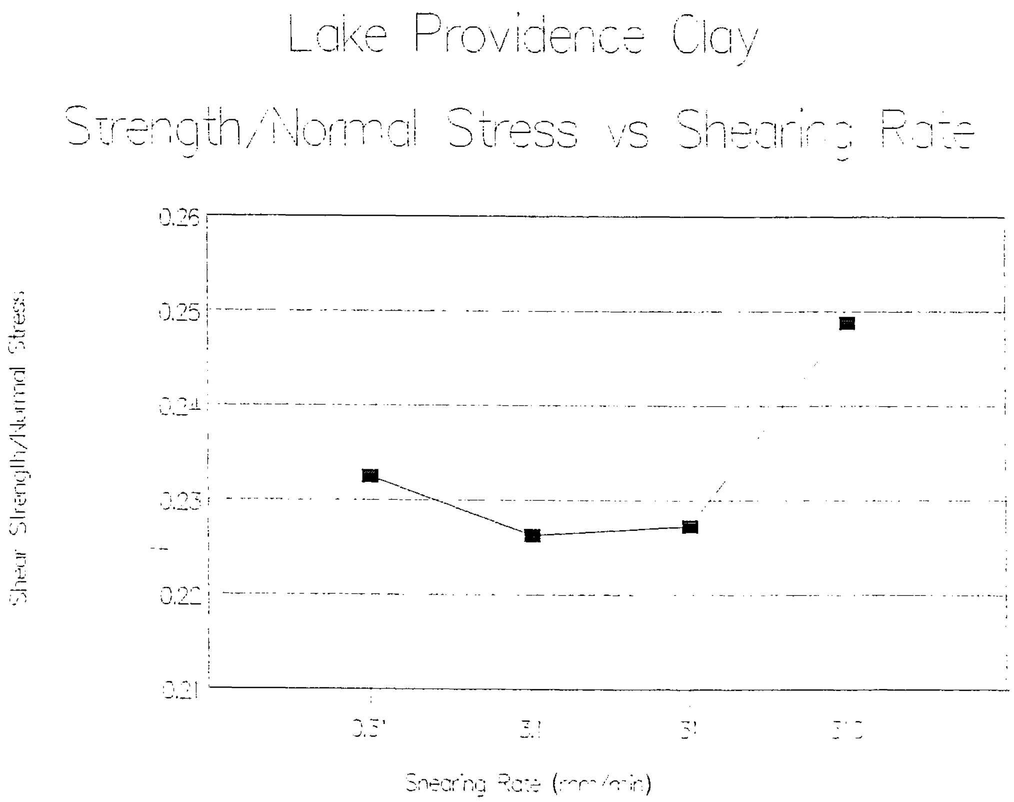



Figure 5.5 Rate Effect Results by Hvorslev & Kaufman (1951)

Ring shear tests at two different shearing rates were conducted by de Beer (1967) on Boom Clay. The apparatus was designed for a 18mm thick sample to be sheared at mid-depth and enabled the failure surface to be pre-cut. The tests were conducted at normal stresses from 50 to 250 kPa and at shearing rates of 0.0066 and 0.035 mm/min. The strength was observed to be slightly greater at the slower shearing rate indicating a reverse rate effect.

La Gatta (1970, 1971) reports residual shear tests conducted in a custom made ring shear apparatus. By means of a water bath, the machine provided for the testing of remoulded samples of soil in the presence of moisture; an axial load supplied a normal stress. The annular sample of soil had an outside diameter of 71 mm, an inside diameter of 51 mm, and was 2mm thick prior to consolidation. A sample of Pepper Shale was tested at a normal stress of 100 kPa at rates of 0.0048 and 0.048 mm/min and exhibited a weak reverse rate effect. A small conventional rate effect was observed for a sample of crushed Cucaracha Shale tested at 800 kPa and at rates ranging from 0.0048 to 0.48 mm/min. Similar results were observed for Blue London Clay at 400 kPa over a range of tested rates from 0.0048 to 2.4 mm/min. Over the same shearing rates, and at a normal stress of 400 kPa, Bearpaw Shale showed a large amount of variation in strength but indicated a reverse rate effect. La Gatta concluded that any changes in strength from variations in shearing rate were minimal.

A ring shear testing machine was also developed by Bucher (1975) at the Institute of Foundation Engineering and Soil Mechanics, the University of Zurich. The apparatus provided for the shearing of a 16 mm thick annular sample with outside and inside diameters of 240mm and 160mm respectively. The sample was sheared at mid-height but, unlike the Bishop apparatus, the device to adjust the gap size is mounted to the upper platen and so the normal stress is not affected. While a large number of tests are reported only one soil was tested to study rate effects. Sowerbyi Clay was tested at normal stresses of 78 and 177 kPa over shearing rates from 0.015 to 15 mm/min and a small conventional rate effect was observed.

Salt (1988b) reported a series of ring shear tests on a clayey silt conducted in a Bromhead type apparatus. The work was done as part of an investigation into the stability of creeping landslides in the Cromwell Gorge in the South Island of New Zealand, where a major hydro electric reservoir was to be constructed. The tests were conducted at normal stresses from 240 to 1100 kPa, and indicated conventional rate effects from rates between 0.00002 and 0.3 mm/min.

Smith and Salt (1988) reported tests which were carried out on East Abbotsford Clay which was acquired from the 1979 East Abbotsford landslide in Dunedin, New Zealand. The tests were conducted at a normal stress of 500 kPa at shearing rates between 0.0007 and 7 mm/min, and over the entire range the sample exhibited conventional rate effects.

A modified Bromhead ring shear apparatus was used by Boyce et al (1988) to study the residual strength of Lias Clay. Modifications to the ring shear apparatus involved the installation of vanes in the shearing cell to ensure the sample shears at mid-depth (described in Anayi et al, 1989a). The sample was sheared at normal stresses ranging from 25 to 600 kPa and at rates between 0.017 and 0.17 mm/min. Negligible rate effects were observed over the two rates.

Table 5.6 summarises the ring shear tests presented in this section and includes the results observed in this research project. The only data at fast rates, other than those obtained by Imperial College, are those from the pioneering work of Hvorslev and Kaufman (1951) and the results in this thesis. Both these two latter results indicate positive rate effects. At slow rates both conventional and reverse rate effects are observed. The results suggest that soils that are dominantly clay or silt tend to exhibit conventional rate effects, while the intermediate soils that have significant fractions of both particle sizes tend to show reverse rate effects.

### 5.5.4 Discussion of Cohesive Soil Rate Effects

A large number of researchers have studied the rate effects for cohesive soils. The results reviewed in this chapter cover a wide range of soils and involve tests in many different types of testing apparatus.

Table 5.6 Summary of Rate Effect Studies on Cohesive Soils by other Researchers

| AUTHOR(S) | SOIL DESCRIPTION | %CLAY FRACTION | NORMAL STRESS | SHEARING RATES mm/min | RATE EFFECTS | |
|---|---|---|---|---|---|---|
| | | | | | Slow Rates | Fast Rates |
| Hvorslev & Kaufman (1951) | Lake Providence Clay | - | 196 kPa | 0.31 - 310 | Reverse | Positive |
| de Beer (1967) | Boom Clay | 49 % | 49 - 245 kPa | 0.0066 - 0.035 | Reverse | - |
| La Gatta (1970) | Pepper Shale | - | 100 kPa | 0.0048 - 0.048 | Reverse | - |
| | Cucaracha Shale | 42 % | 800 kPa | 0.0048 - 0.48 | Conventional | - |
| La Gatta (1971) | Blue London Clay | 57 % | 400 kPa | 0.0048 - 2.4 | Conventional | - |
| | Bearpaw Shale | 53 % | 400 kPa | 0.0048 - 2.4 | Reverse | - |
| Bucher (1975) | Sowerbyi Clay | - | 78 - 177 kPa | 0.015 - 15 | Conventional | - |
| Salt (1988b) | Cromwell Clayey Silt | 22 % | 240 - 1100 kPa | 0.00002 - 0.3 | Conventional | - |
| Smith and Salt(1988) | East Abbotsford Clay | 35 % | 500 kPa | 0.0007 - 7 | Conventional | - |
| Boyce et al (1988) | Lias Clay | - | 25 - 600 kPa | 0.017 - 0.17 | Negligible | - |
| Smith (1990) | Temuka Clay | 52 % | 50 - 400 kPa | 0.00029 - 2079 | Reverse | Positive |

A high proportion of the samples tested showed different behaviour at fast and slow rates. The approximate boundary between the two types of rates seems to rest somewhere in the range from 0.1 to 30 mm/min.

At slow rates of shear, all the observed rate effects are small. In many cases the observed rate effects are smaller than the accuracy of the data recording system and it may be concluded that the rate effects are negligible. The results generally indicate that samples of high clay fraction and high plasticity exhibit conventional rate effects. As the silt fraction increases and the plasticity decreases, soils generally show reverse rate effects.

At fast rates, the study of rate effects is complicated by the loss of soil sample and the generation of pore pressures. It is shown here that many of the reported results at fast rates may have been significantly distorted by this effect. All of the remaining results indicated positive rate effects.

## 5.6 CONCLUSIONS

The effect of the rate of shear on the strength of materials has been the focus of many studies because of its importance in understanding the mechanisms of failure.

Steel behaves as a thixotropic plastic. During testing, it increases in strength with increasing shear rate, but at a rate less than proportional to the strain rate. The changes in strength are small and amount to only a few percent per tenfold change in shearing rate.

The behaviour of rocks has been found to be more complex. At slow rates and in the presence of small amounts of moisture, all the rocks tested show reverse rate effects (velocity weakening). At high temperatures, or when vacuum dried, conventional rate effects (velocity strengthening) are observed. The importance of moisture in determining the type of rate effect is attributed to the part water plays in the time-dependent creep of the surface indentations. At rates in excess of 1 mm/min, the rocks tested consistently show positive rate effects.

The artificial granular materials that have been tested generally show negligable rate effects at both fast and slow rates. At very fast rates and at low normal stresses, some small positive rate effects are observed. These are attributable to the viscous effects of the component fluid.

Cohesionless soils tested over a very wide range of shearing rates show very little sensitivity to the rate of shearing. The majority of tests were unable to identify any rate effect whatsoever. Some tests at very fast rates, and with a high degree of accuracy, identified a weak positive rate effect.

A wide range of behaviour has been reported for cohesive soils, with various tests indicating all of the possible combinations of conventional, reverse, positive and negative rate effects. At slow rates, all the observed effects are small. Conventional rate effects are typically reported for soils with a high clay fraction, while soils with a significant silt fraction show reverse rate effects. At fast rates, many results can be discounted as questionable due to the effects of pore pressures generated by sample loss. The remaining test results for cohesive soils at fast rates show positive rate effects (ie velocity strengthening).

The results of rate effect studies show that most materials increase slightly in strength with increasing shear rate. The exceptions to this rule are rock and some natural soils which weaken with increasing shear rate in the slow range. The explanation for this behaviour is contained in the effects of creep on the mechanics of surface indentation in rock minerals.

# CHAPTER 6

# RATE EFFECTS AND LANDSLIDE STABILITY

## 6.0 INTRODUCTION

Assessing the stability of a landslide is an exceptionally difficult engineering problem. If past movements have occurred, this indicates that the slope is only marginally stable, and that it will be sensitive to small changes in the key variables. Furthermore, small movements of the slide may have significant effects on the strength of the soil which may help stabilise or destabilise the slope.

Factors that influence stability can be categorised as internal or external. External factors are those that change the forces acting on the landslide, and include earthquakes, rainfall events and changes in geometry (man-made or natural). Sufficient knowledge of these factors exists to approximately model their effects. Internal factors are those within the landslide which affect stability by changing the shear strength of the soil. Understanding these factors is critical in determining the stability of a creeping landslide. Internal factors include the initial post-peak drop in strength, pore pressures generated within the failing soil, and rate effects. It is the interaction of the external and internal factors that will determine the long term stability of a landslide.

The primary focus of this research has been rate effects. It was concluded in the previous chapter that rate effects vary with soil type. This chapter will examine the implications of each type of rate effect for landslide stability. Other internal factors that may influence landslides will be discussed, allowing the findings of this research to be reviewed in the light of observed landslide behaviour.

## 6.1 STABILITY IMPLICATIONS OF THE OBSERVED RATE EFFECTS

The rate effects observed and reported in this thesis have all been derived from rate-controlled shear tests. The shearing rate has been determined, and the free variable has been the shear strength, which has been measured.

In the natural landslide the opposite is the case. The shear stress is determined by the forces acting on the slide and the shearing rate is the free variable. If the shear strength of the soil is less than the stress required to balance the forces acting on the slide, the slope will accelerate according to Newton's Law. Once a slide is moving, the type of rate effect shown by the soil will have a critical role in determining the slide's future stability.

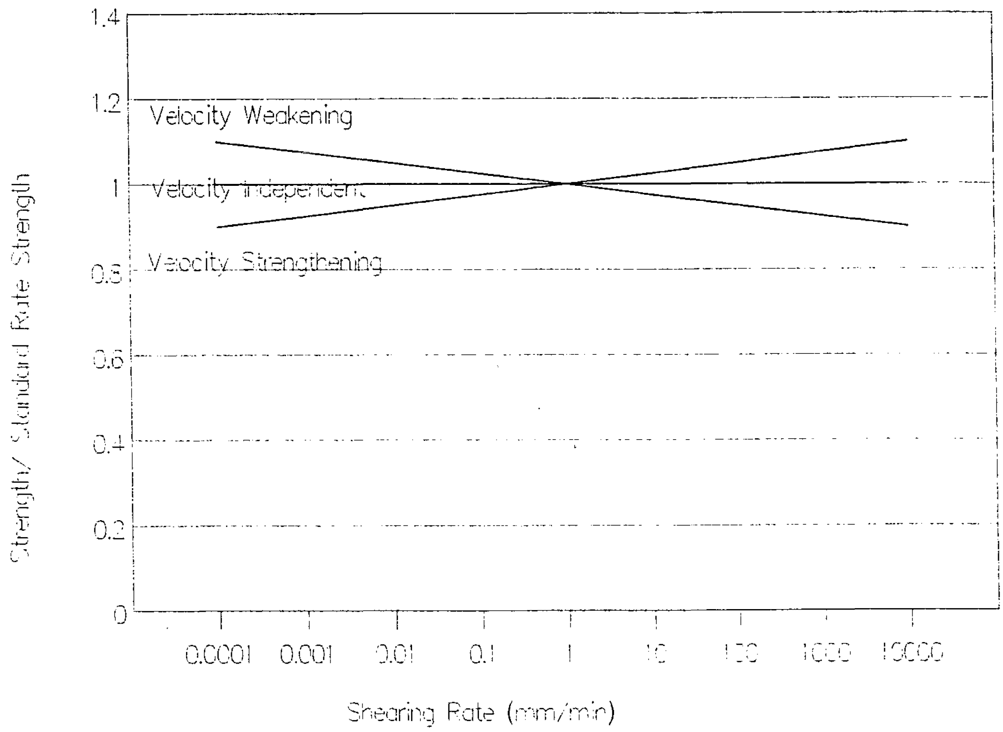



Figure 6.1 Idealised Rate Effect Behaviours

Three types of rate effects for soils are reported in the previous chapter, and are illustrated in Figure 6.1. In this section, the stability implications of each of these rate effects will be considered.

### 6.1.1 Velocity Independent

Most of the cohesionless and artificial soils tested by various researchers show negligible variation in the measured residual shear strength over very large ranges of shearing rate.

In the stress (rather than strain) controlled environment of the landslide, these soils will be quite unstable. Very small increases in stress in excess of residual strength will result in acceleration and failure. Landslides shearing on these types of materials are likely to give only limited warning signs, and are likely to fail at high speeds.

### 6.1.2 Velocity Strengthening

Velocity strengthening includes positive rate effects at fast rates, and conventional rate effects at slow rates. All the cohesive soils reported in this thesis exhibit positive rate effects, and those with greater clay fractions also show conventional rate effects.

A soil that strengthens with increased shearing rate will behave more stably in the stress-controlled situation than those soils that exhibit velocity weakening or negligible rate effects. As the shear stress increases within the failure zone, and exceeds the static residual strength, the slide will accelerate. At the faster shearing rate, the soil will increase in strength and reduce the accelerating force. These mechanisms will interact to establish a stable rate of shearing, at which point the shear strength of the soil summed over the slide area will equate to that required to balance the external forces on the slide.

It is possible to model the effects of changing external factors on a landslide where the failure soil exhibits velocity strengthening, and the strength - velocity function is known. This could have practical applications in estimating the reduction in creep as a consequence of remedial works; and in estimating the increase in creep as a consequence of particular construction works.

### 6.1.3 Velocity Weakening

The term velocity weakening describes soils that exhibit reverse rate effects at slow rates or negative rate effects at fast rates. In the previous chapter, it was found that most cohesive soils show reverse rate effects. Negative rate effects have only been observed in tests where it is likely that pore pressures have distorted measurements.

Soils that exhibit velocity weakening will be inherently unstable in the stress controlled environment of a natural landslide. If the shear stress exceeds the static shearing resistance, the slide will begin to move, and the shear strength will reduce. This will cause further acceleration, and the unstable mechanism will either develop into catastrophic failure, or end as a result of a switch to velocity strengthening at a faster rate.

Results reported in this thesis (and not subject to experimental problems), all show reverse rate effects changing to positive rate effects at rates in excess of 1 mm/min. This suggests that once movement has been initiated, it would accelerate to rates in excess of 1 mm/min. Stable sliding at a rate less than 1 mm/min would not be possible. Movement would be either rapid or non-existent.

The reported tests on rocks showed these rate effects, and so it would be expected that the failure of natural rock interfaces would occur unstably. The sudden stick-slip behaviour of faults, as manifested in earthquakes, is evidence of this.

Applying this rate effect data to landslides is more difficult. This is because it is assumed that displacements and shearing velocities occur simultaneously on all parts of the shear interface. The rigidity of rock makes this a fair assumption in analysing the behaviour of faults, but soils are more elastic. Displacements may vary slightly within a landslide due to the elastic deformation of the soil mass. Qualitatively, it would be expected that different segments of the landslide would move in a stick-slip manner. Each movement would propagate in the form of a wave and the landslide would, in effect, slowly shuffle down the slope.

A full analysis of the effects of the observed reverse/ positive rate effect on landslide stability would require the development of a numerical model that allowed each segment of the landslide to move independently. This is beyond the scope of this thesis, but would be a worthwhile topic for future research.

## 6.2 OTHER INTERNAL FACTORS INFLUENCING STABILITY

The rate effects have an important role in determining the stability of a landslide, but other factors are also influential. These include changes in shear strength with displacement, pore pressures generated within the failing soil, and geometric effects of landslide movement.

In the case of first-time landslides, the variation in strength as a consequence of displacement is very important in determining stability. Most soils exhibit a post-peak drop in strength when initially sheared. In overconsolidated clays, the strength may reduce by 50% of the peak value in the first 100 mm of displacement, and this could trigger catastrophic failure. For landslides that have already undergone significant displacements, the soil strength is not dependent on displacement, and this factor is therefore unimportant.

The effects of pore pressures on slope stability are well understood. An increase in the pore pressure within the failure zone decreases the effective normal stress and the shear strength of the soil. Pore pressures generated by movement of the slide can have a significant effect on stability. During initial shearing of a first-time slide, the change in moisture content as the failing soil approaches its critical state can generate pore pressures. These can be approximated using Skempton's pore pressure parameter A

(Lambe & Whitman, 1979), but are only relevant for first-time slides. This concept is extended by Bernander et al (1985) for first-time landslides in sensitive soils.

An alternative source of pore pressures affecting both first-time and ancient landslides has been postulated. It is suggested that significant pore pressures may be generated by internal frictional heating, as a consequence of slide movement. An examination and numerical analysis of the effects of this phenomenon was undertaken by the author, in conjunction with other New Zealand landslide researchers. The study concludes that pore fluid pressure due to heat-induced expansion may enhance creep rates and possibly lead to total loss of stability. (Davis et al 1990).

A further factor that may influence stability is a change in the geometry of a landslide as a consequence of displacement. Where a landslide is sited on a constant slope, this factor is not significant, but some slopes may increase or decrease in gradient. In the case of a concave slope, downward movement will enhance stability. On the other hand, a convex slope will cause a landslide to accelerate once moving.

## 6.3 STABILITY OF HISTORICAL LANDSLIDES

It is useful to conclude this thesis with a discussion of historical landslides, and to review the findings of rate effect studies in the light of observed behaviour.

There are few examples of landslides where the failing soil has been extensively tested in the laboratory for rate effects. In this section, three such landslides are considered. One other landslide is also reported because of the unique shearing mode that was observed.

### 6.3.1 East Abbotsford Landslide

The East Abbotsford Landslide occurred on the 8th August, 1979, in Green Island, a suburb of Dunedin, New Zealand (See Photograph 1.1 - Page 2). Prior to rapid failure, the slope had shown signs of movement as early as 1969, and by the evening of the 8th August, 1979, had already displaced by nearly 2 metres. That evening, the mass of soil, 30 m in depth, and 5 million $m^3$ in volume, moved 50m downslope in approximately 30 minutes. (Coombs and Norris, 1981).

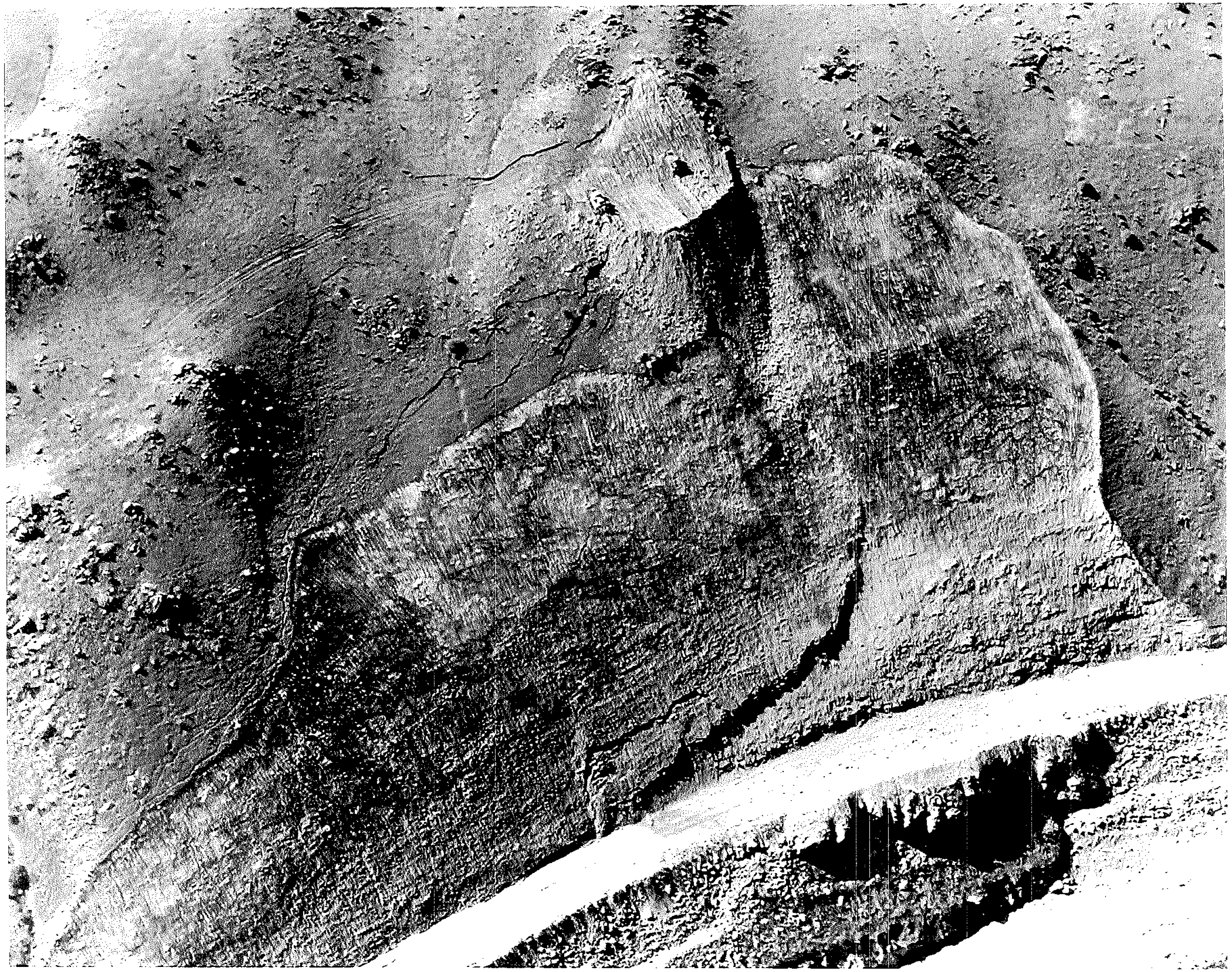
Failure occurred on a thin layer of highly over consolidated clay with a residual friction angle of 9.5° (Salt et al 1980). It can be assumed that the soil was at residual conditions because a significant displacement had occurred prior to final failure. As part of a study into the mobility of the Abbotsford landslide, a series of ring shear tests was undertaken to determine the rate effects of the soil. In varying the shearing rate from 0.0007 to 7 mm/min, conventional and positive rate effects (i.e. velocity strengthening) were observed. Over this range the strength increased by 8 % (Smith & Salt, 1988).

External factors affecting the stability of the landslide included deforestation, antecedent rainfall, water main leakage, and excavation of a borrow pit. By analysing the effects of each of these mechanisms on the factor of safety, it is possible to estimate the subsequent changes in shearing rate. A quantitative analysis of this kind, by Smith & Salt (1988), shows a reasonable comparison between predicted and actual behaviour.

Qualitatively, the Abbotsford landslide developed slowly, and even during failure did not exceed a velocity of 3 m/min. This is consistent with the behaviour postulated earlier in this chapter for a velocity strengthening soil.

### 6.3.2 Brewery Creek Landslide

A number of ancient landslides have been identified in the Cromwell Gorge in Central Otago, New Zealand, in the course of the hydro-electric development of the Clutha River. The Brewery Creek landslide (see Photograph 6.1) is located on the east bank of the Clutha River, 3 km south of the township



Photograph 6.1 Brewery Creek Landslide

of Cromwell. The slide consists of 10 million $m^3$ of disturbed schist and slide debris (Salt, 1985a).

During construction of a road, shown in the bottom right corner of the photograph, the slide was reactivated. Stabilisation was attempted by dozing material from the head of the slide to a point downslope. The earthmoving machinery and material are visible in the aerial photograph. Work progressed for five days, until it became apparent that a new failure surface had been activated downslope. Over the following five days, the material was moved laterally off the lower slide, during which time displacements were measured at 12-hour intervals.

Clayey silt from the site area was tested in the ring shear apparatus by Salt (1988b), as reported in Section 5.5.3. The tests identified conventional rate effects over a range of 0.00002 to 0.3 mm/min, with an increase of approximately 2 % in strength for each ten fold increase in shearing rate. (See Figure 6.2)

A limit equilibrium analysis of the slope shows that the removal of material over the five-day period would result in an increase in the factor of safety by 3 %. It is assumed that this occurs at a constant rate over the five days, i.e. a decrease in the shearing stress required to maintain equilibrium of 0.6 % per day. Noting that the shearing rate of the slope when under maximum load was 0.5 mm/min (700 mm/day), it is possible, using the ring shear data in Figure 6.2, to predict the reduction in the shearing rate.

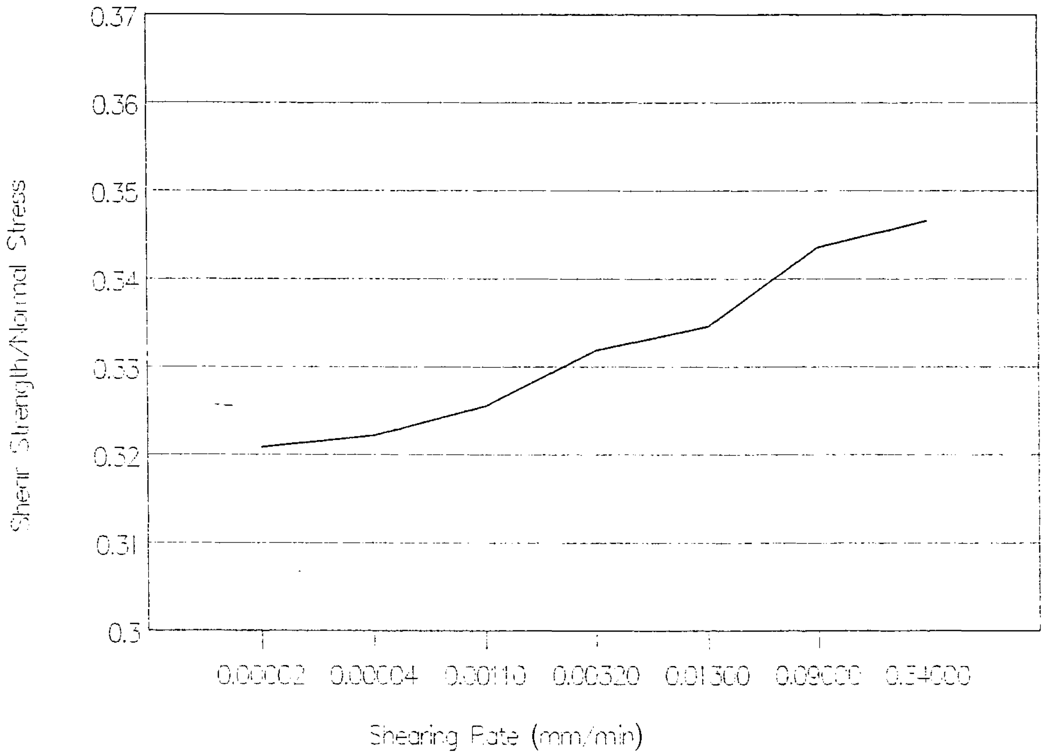



Figure 6.2 Rate Effects for Cromwell Clayey Silt

These predictions and the actual shearing rate of the slide, determined from the measured displacement of three pegs located on the moving slide, are shown in Figure 6.3. The similarity of the two curves confirms the relevance of the rate effect data from ring shear testing to actual landslide movements.

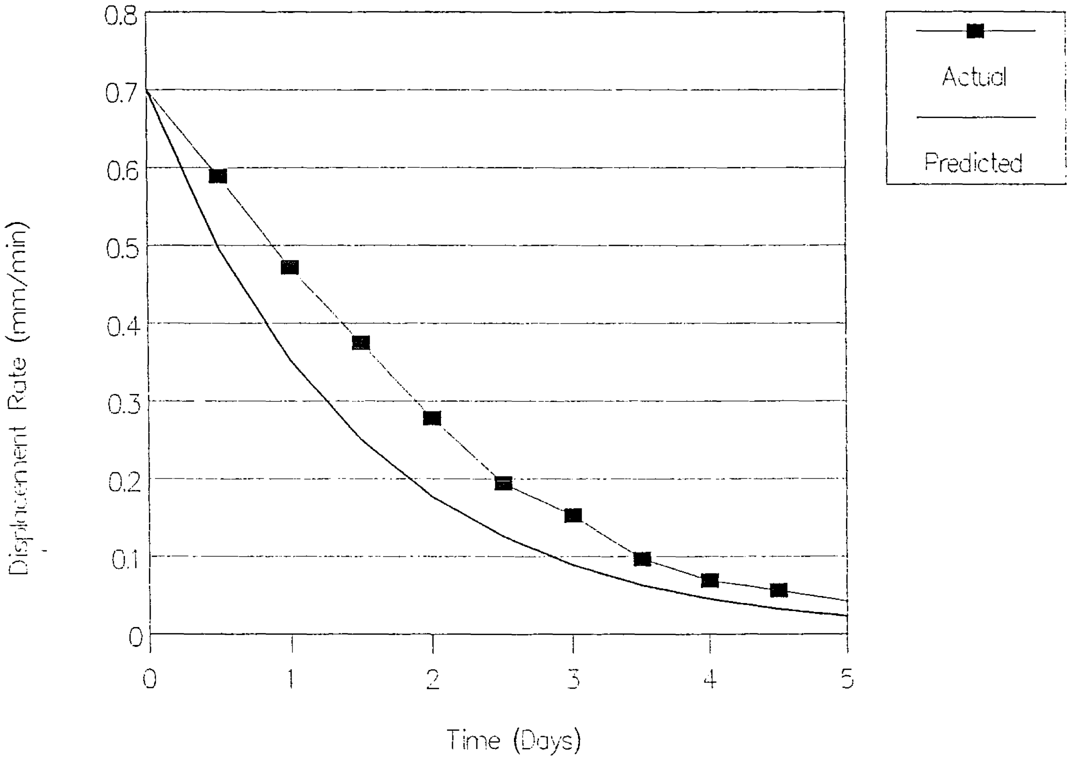



Figure 6.3 Comparison of Actual and Predicted Displacement Rates for Brewery Creek Landslide

6.3.3 Mt. Ontake Landslide

The Nagoken - Seibu earthquake in 1984 triggered a large landslide on the south east flank of Mt. Ontake, Japan. The 80 metre thick landslide consisted of 36 million $m^3$ of rock and debris, which flowed down the mountain and valley for over 9 km, at an average velocity of 20 - 26 m/s. (Sassa, 1987).

The failing material was a wind-blown yellowish pumice deposit, which would have been produced as a consequence of volcanic activity in the area. The pumice was identified by Tika (1989) as a sandy silt composed of highly porous halloysite. Ring shear tests by Tika (1989) to study rate effects (reported in Section 5.5.2 of this thesis), showed the soil to have negligible rate effects over a shearing range from 0.01 to 6000 mm/min. Similar high speed ring shear tests by Sassa (1987) at rates between 0.6 and 60,000 mm/min also showed negligible rate effects.

The conditions at failure of this slope differ from both the Abbotsford and Brewery Creek landslides, in that the activating mechanism is more sudden. However, it is also significant that the material showed negligible rate effects in the ring shear apparatus. The slope would have been very sensitive to changes in loading and would behave unstably once motion had been initiated.

### 6.3.4 Rainbow Avenue Landslide

The Rainbow Avenue landslide occurred in February, 1969, in the Mt. Washington area of Highland Park in Los Angeles County, California. The slide, about 45 m wide and 150 m in length, had moved during the 1952 Tehachapi earthquake. Movement was reactivated during a heavy rain storm in January, 1969, as a consequence of erosion of toe material and increased saturation of the slope. (Scott, 1978).

The monitoring of this landslide undertaken by Scott was unique, in that very accurate instrumentation was used which enabled movements as small as 0.0025 mm to be detected. The measurements revealed that the landslide was not moving continuously, but was instead shuffling downslope in small regular amounts. Initially these movements were about 0.25 mm in magnitude, and occurred once every 15 minutes. This slowed over a two week period to 0.13 mm every 45 minutes. The magnitude of the movement was confirmed with daily surveys of marker pegs. The unusual behaviour was further studied by locating a similar measurement device 15 m downslope. Identical behaviour was noted except that the movement preceded the higher monitoring point by approximately 1 minute. This indicated that a wave movement was propagating up the landslide.

The failing soil was a montmorillonite clay formed from the weathering of Miocene and Pliocene sedimentary shales. No ring shear study of the rate effects of the soil was undertaken. However, other montmorillonite shales tested, as reported in Chapter 5, show reverse rate effects at slow rates. In an idealised rigid slope mass, reverse rate effects would be expected to cause movements to accelerate until positive rate effects stabilised the slope. A possible explanation of the Rainbow Avenue landslide is that the internal flexibility of the landslide enabled elements of the slide to move independently in a stick-slip mode caused by reverse rate effects. Confirmation of this possible explanation would require the clay shale to be tested for rate effects and the results used in a landslide model that allowed segments of the landslide to move independently.

## 6.4 CONCLUSIONS

An important factor in determining the stability of a landslide is the sensitivity of the slope to changes in external loads. Ring shear tests over a wide range of shearing rates enable this sensitivity to be ascertained.

In the case of a soil that shows negligible change in strength with shearing rate, the landslide will be very sensitive to any changes in loading. Landslides in these types of soils will tend to fail catastrophically and without much warning.

Soils that exhibit increases in strength with increased shearing rate are more stable. Small increases in external loads can be accommodated with stable increases in the rate of movement of the slide. Crude estimation of the changes in shearing rate in response to external loads are possible, and may be of practical use in determining the cost effectiveness of landslide stabilisation works.

Reductions in soil strength with increased shearing rate will result in unstable shearing. However, such an effect was only observed in tests of soils at slow rates, and switched to velocity strengthening at fast rates. An increase in external loads will cause landslides failing in soils of this type to accelerate at slow rates until velocity strengthening occurs.

Cohesionless soils consistently show negligible rate effects while cohesive soils show both velocity strengthening and weakening. Until sufficient date exists to indirectly identify rate effects from soil classification, extensive ring shear testing will be necessary to determine landslide stability.

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# APPENDIX I

## SOIL DESCRIPTION

Soil Name : Temuka Clay

Unified Classification : CL

Colour : Light Brown

Description : Silty Clay

Liquid Limit : LL = 38

Plastic Limit : PL = 23

(See Determination of Atterberg Limits)

Plasticity Limit : PI = 15

Clay Fraction : CF = 52%

(See Determination of Grain Size Distribution)

Activity : $A_c$ = 0.29

Permeability : $k_s = 4 \times 10^{-10}$ m/s

(See Determination of Permeability)

## Determination of Grain Size Distribution

A sedimentation test was undertaken to determine the grain size distribution of the soil. The test was conducted as per New Zealand Standard NZS4402, Part 1 : 1980, Test 9 (D).

Approximately 60 grams of the soil was mixed with Calgon in the sedimentation cylinder and tested over 3 days. Figure A3.1 shows the determined grain size distribution.

## Determination of Atterberg Limits

Atterberg limits of Temuka Clay were determined according to New Zealand Standard NZS 4402, Part 1 : 1980, Test 2 and 3. The plastic limit was determined from the moisture content test of 20g of soil prepared at its plastic limit. The liquid limit was derived from the graphical method using five samples at different moisture contents.

## Determination of Permeability

The permeability test was conducted on a 15mm sample of clay overconsolidated in the odeometer. The falling head method of determination was used over a three hour period during which very accurate burette readings were regularly taken. The permeability was determined from the area of the sample, the head and measured flow using Darcy's equation.

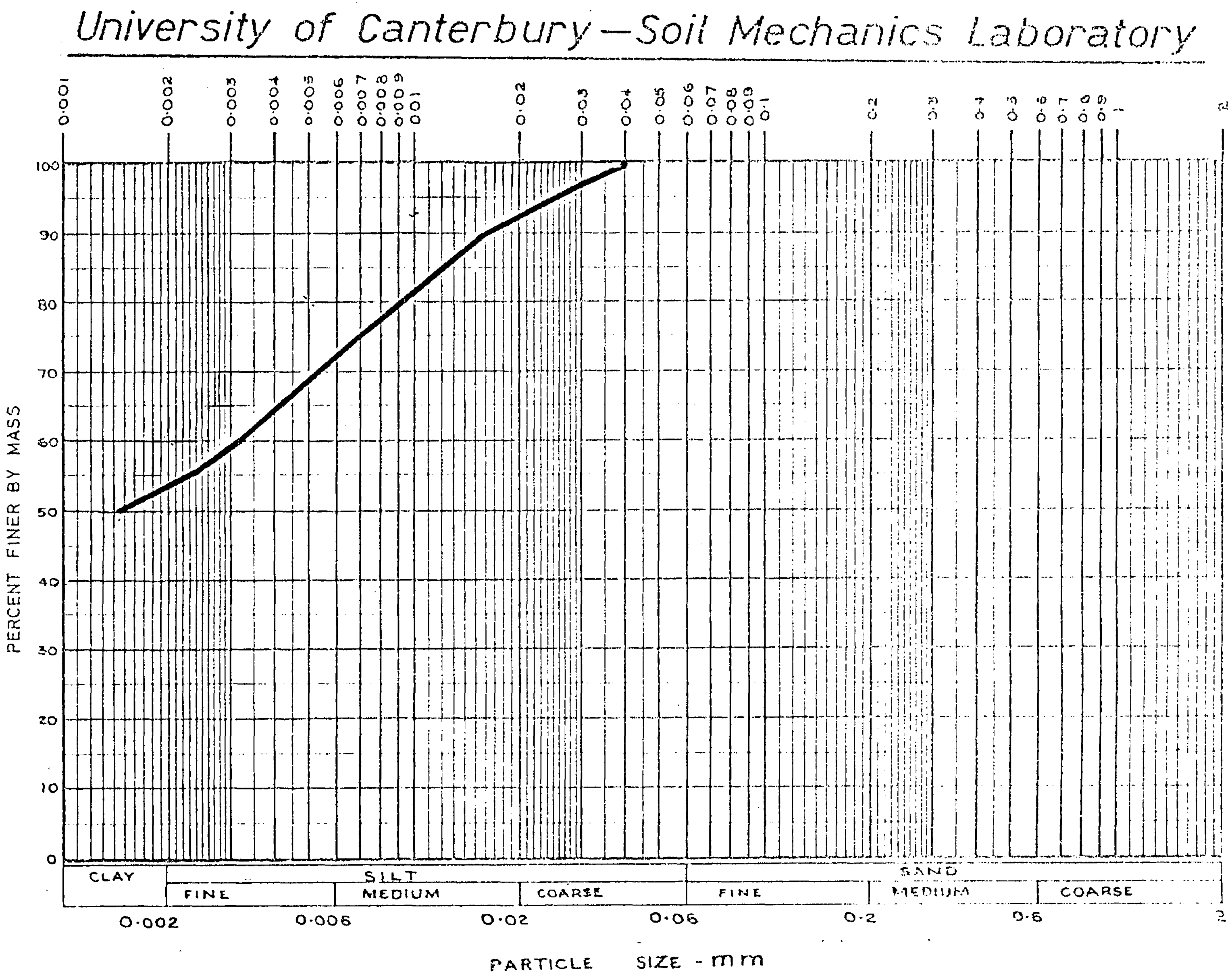



Figure A1.1 Temuka Clay Grain Size Distribution

# APPENDIX II

## OEDOMETER TEST

Soil : Temuka Clay

Batch : 4

Date of Test : June 1988

Oedometer : Machine 3 , Geomechanics Laboratory, U. of C.

Initial Moisture Content : 40.5 %

Final Moisture Content : 29.8 %

| STAGE | NORMAL STRESS | SAMPLE DEPTH | VOID RATIO e |
|---|---|---|---|
| 0 | 1 kPa | 18.350 | 1.073 |
| 1 | 9.1 kPa | 18.016 | 1.038 |
| 2 | 17.4 kPa | 17.642 | 0.994 |
| 3 | 30.0 kPa | 16.990 | 0.920 |
| 4 | 50.0 kPa | 16.662 | 0.888 |
| 5 | 100.0 kPa | 16.396 | 0.855 |
| 6 | 175.0 kPa | 16.051 | 0.819 |
| 7 | 300.0 kPa | 15.603 | 0.768 |
| 8 | 100.0 kPa | 15.719 | 0.778 |

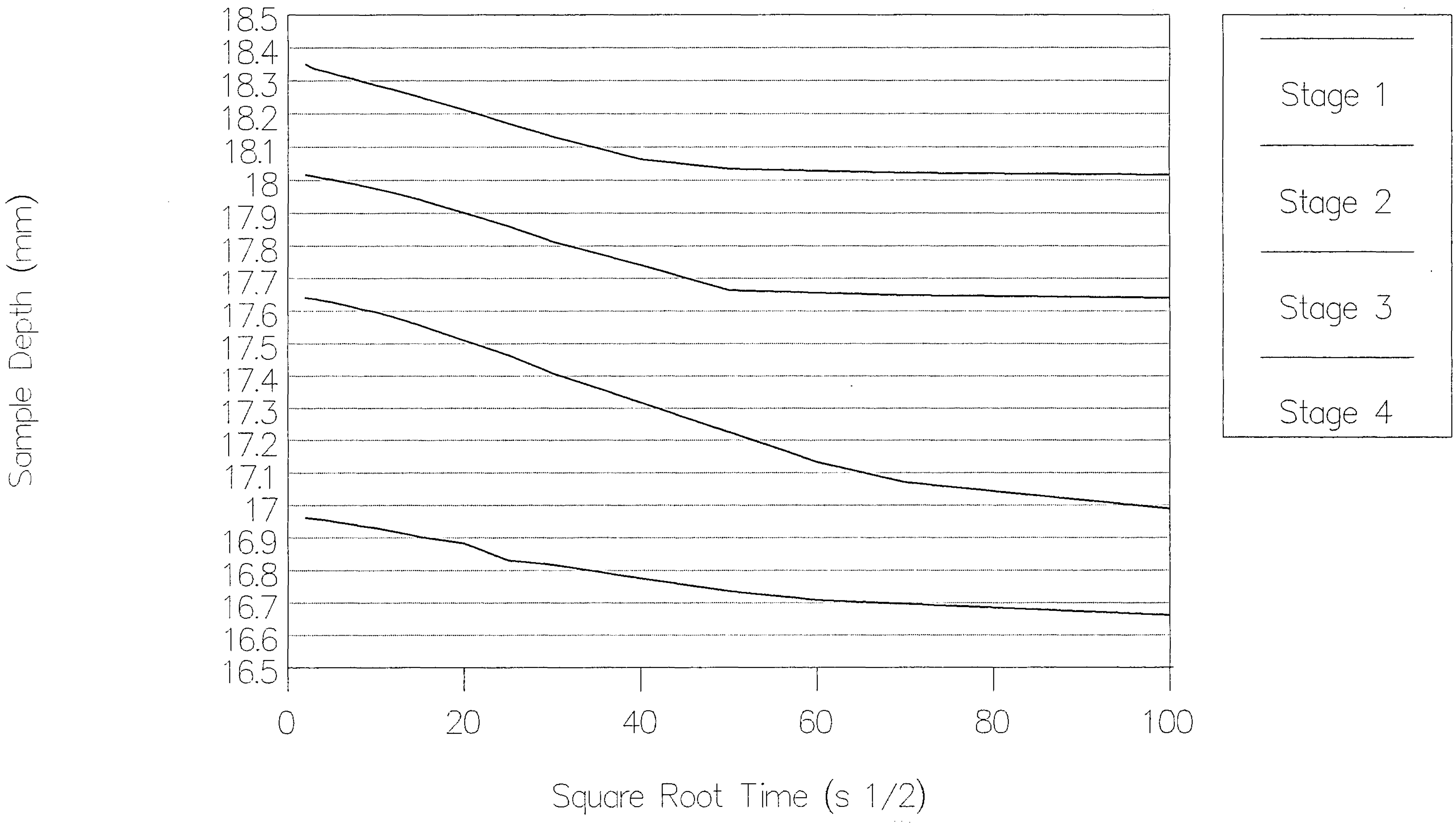



Figure A2.1 Oedometer Test Stages 1 - 4

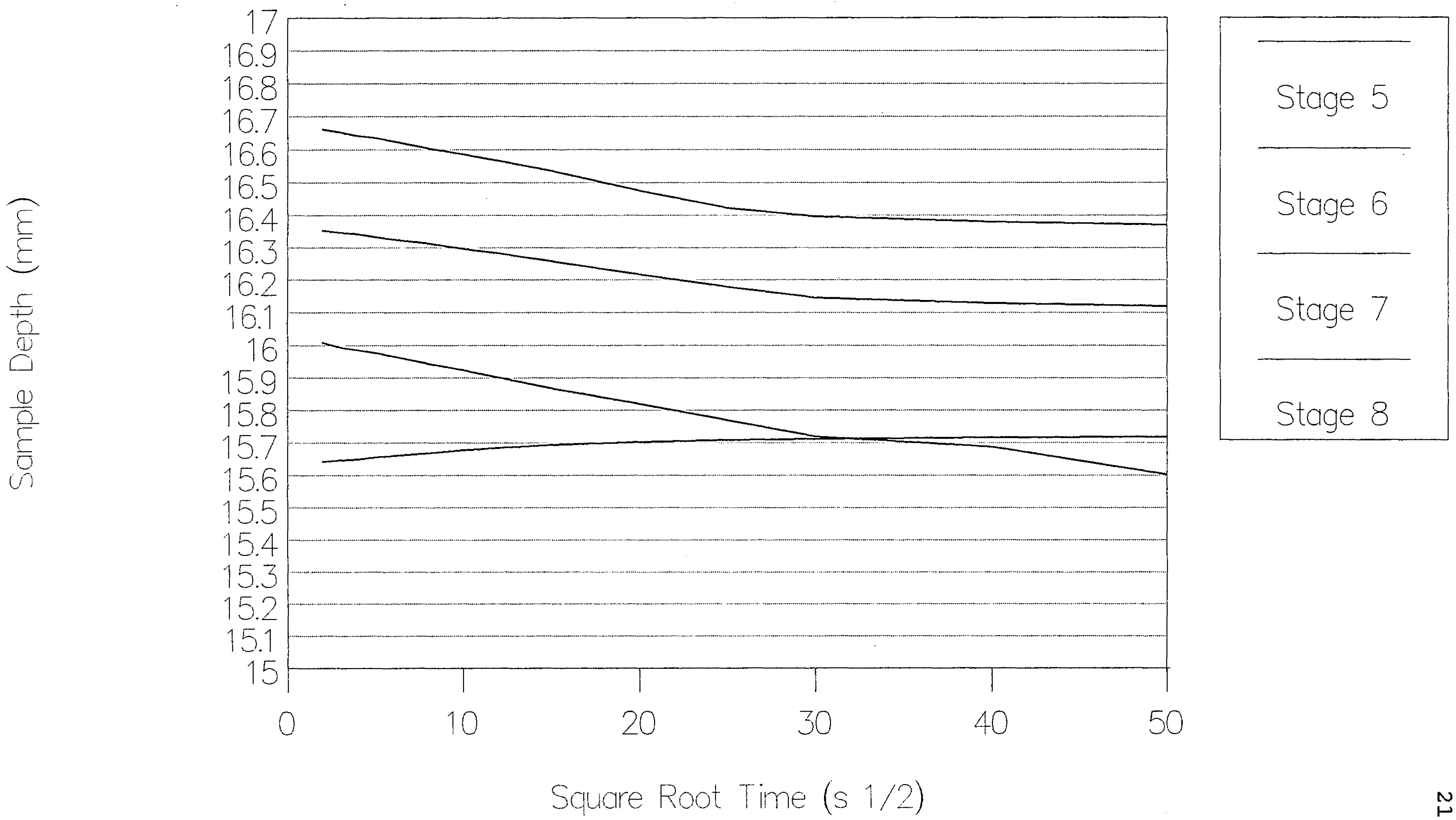



Figure A2.2 Oedometer Test Stages 5 - 8

# APPENDIX III

## DIRECT SHEAR TESTS

DIRECT SHEAR TEST # 1

Soil : Temuka Clay

Batch : 5

Date of Test : 1st February 1990

Normal Stress : 100 kPa

Overconsolidation Ratio : 3

Initial Sample Depth : 8.250 mm

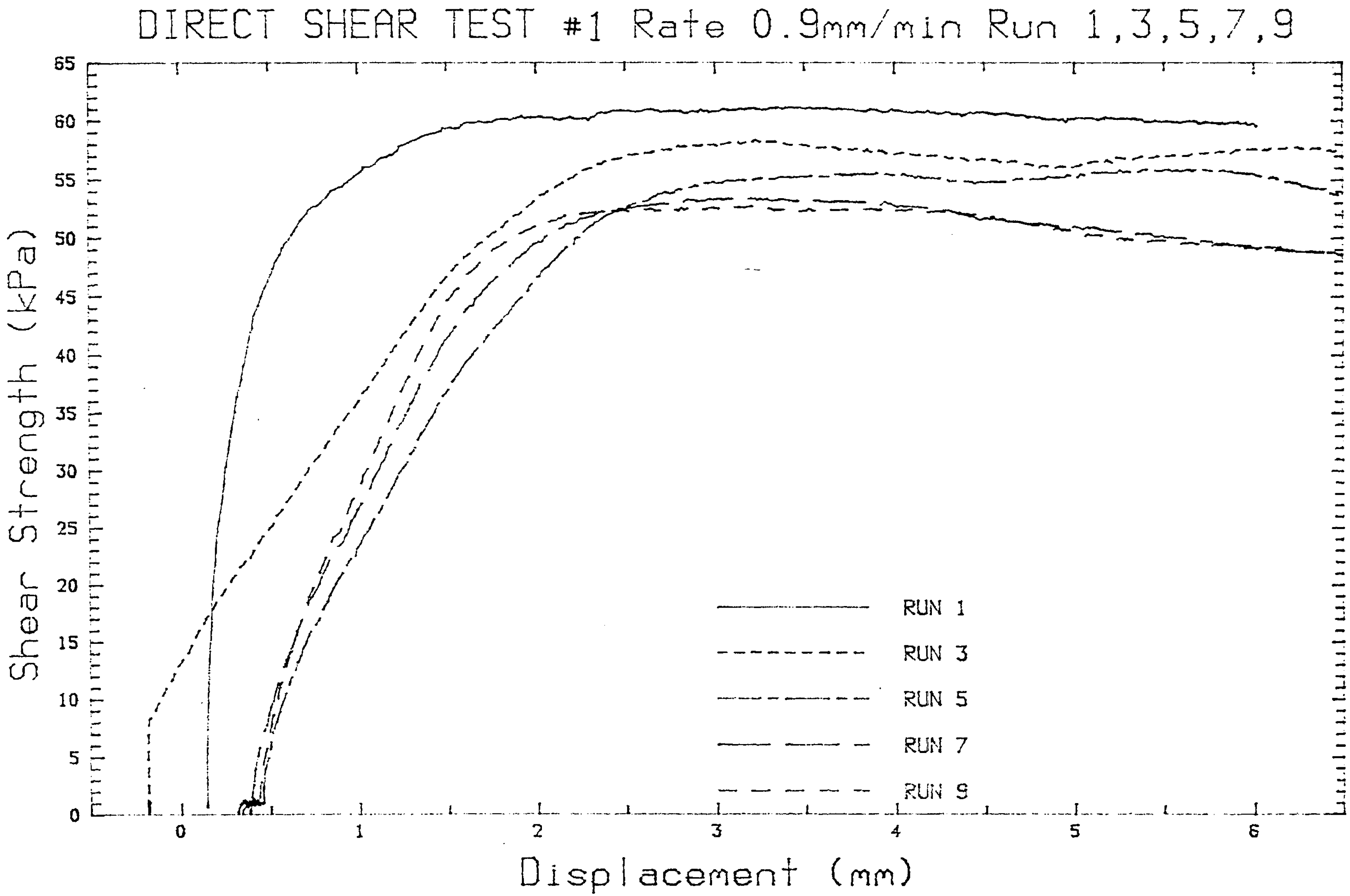
Consolidation Stages

| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 21 kPa | 7.882 mm |
| 2 | 48 kPa | 7.383 mm |
| 3 | 95 kPa | 7.069 mm |
| 4 | 188 kPa | 6.795 mm |
| 5 | 300 kPa | 6.585 mm |
| 6 | 100 kPa | 6.691 mm |

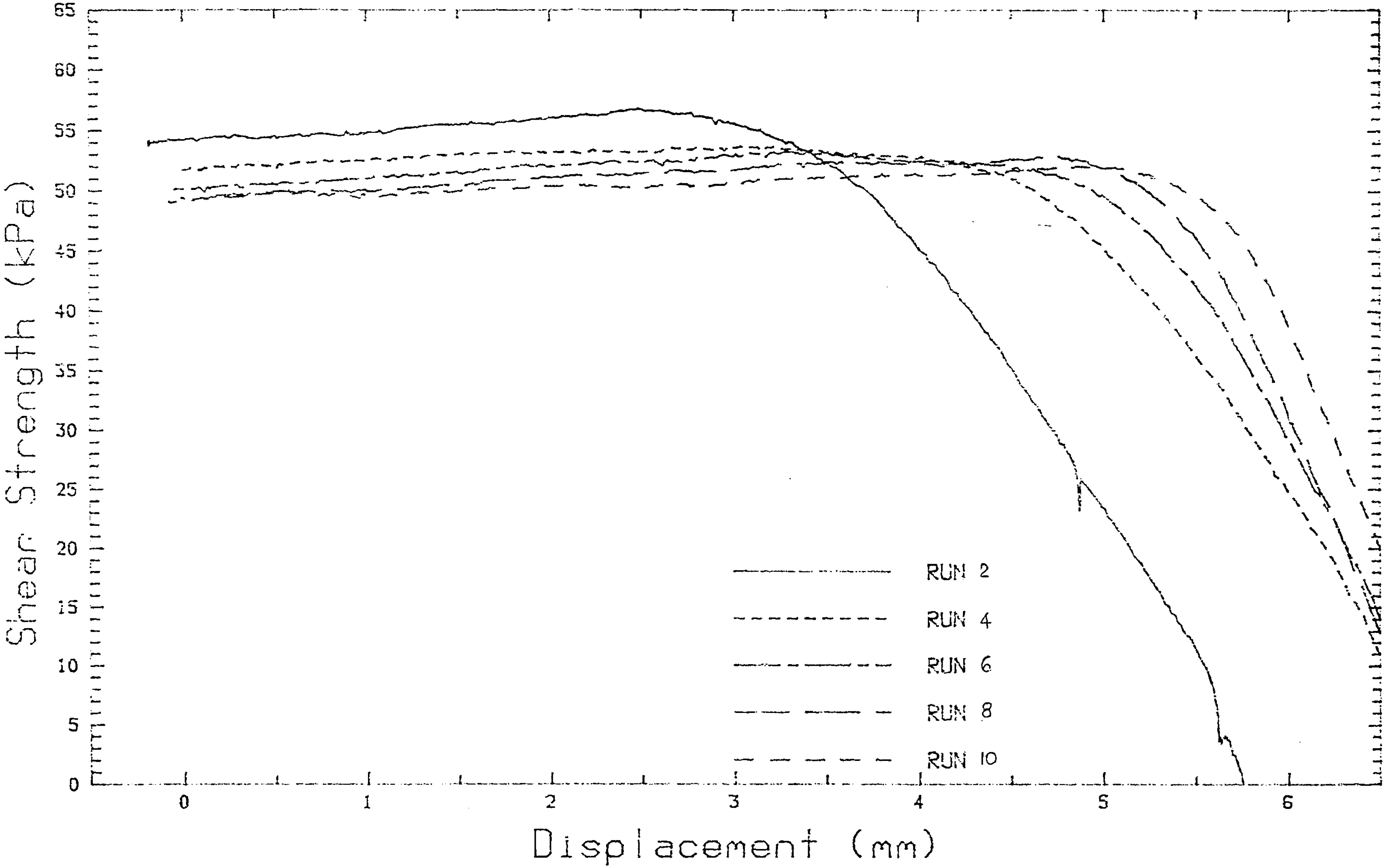
Shearing Stages

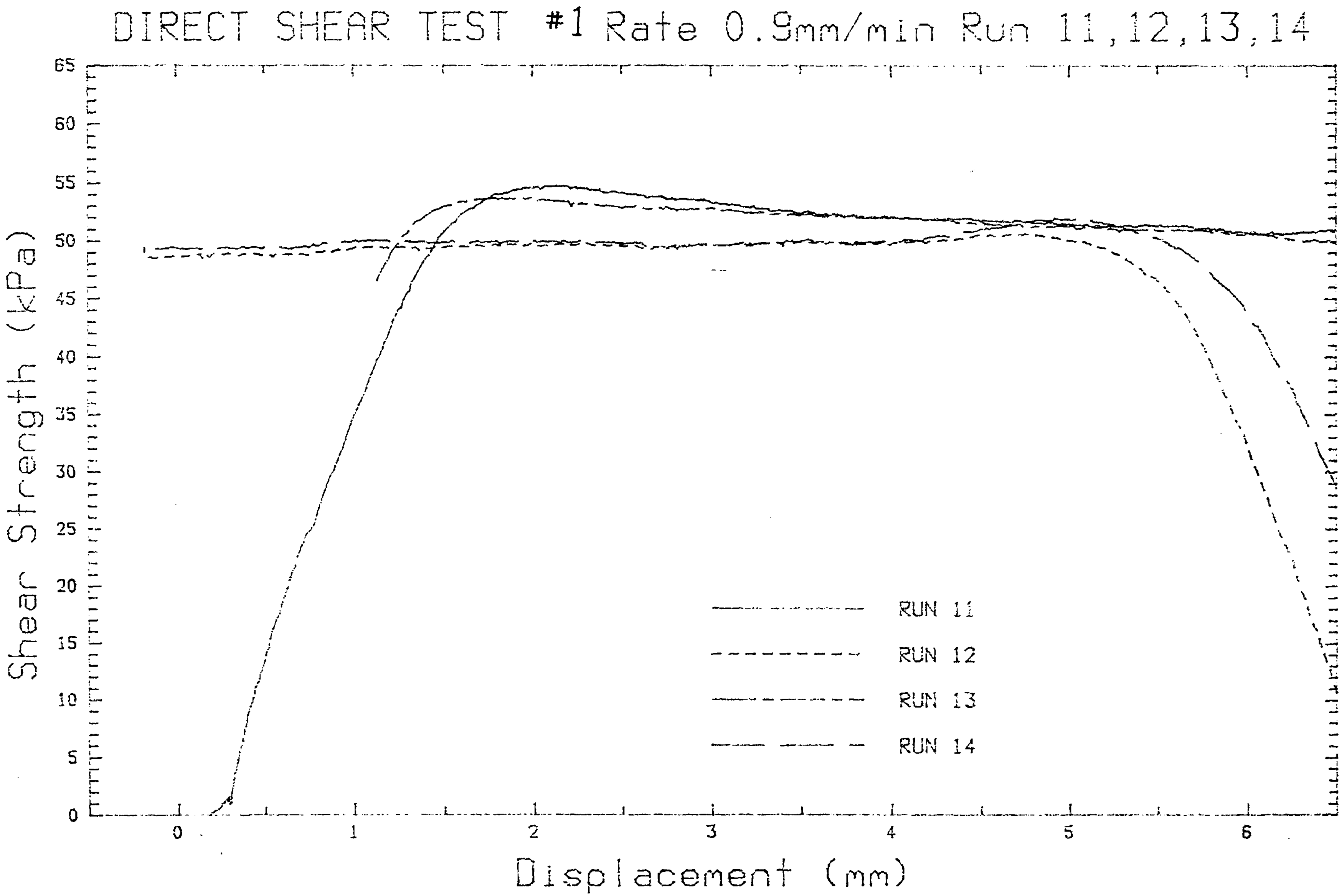
| STAGE | SHEARING RATES |
|---|---|
| 1 - 14 | 0.9 mm/min |
| 15 - 16 | 0.12 mm/min |

Final Moisture Content : 26.7 %

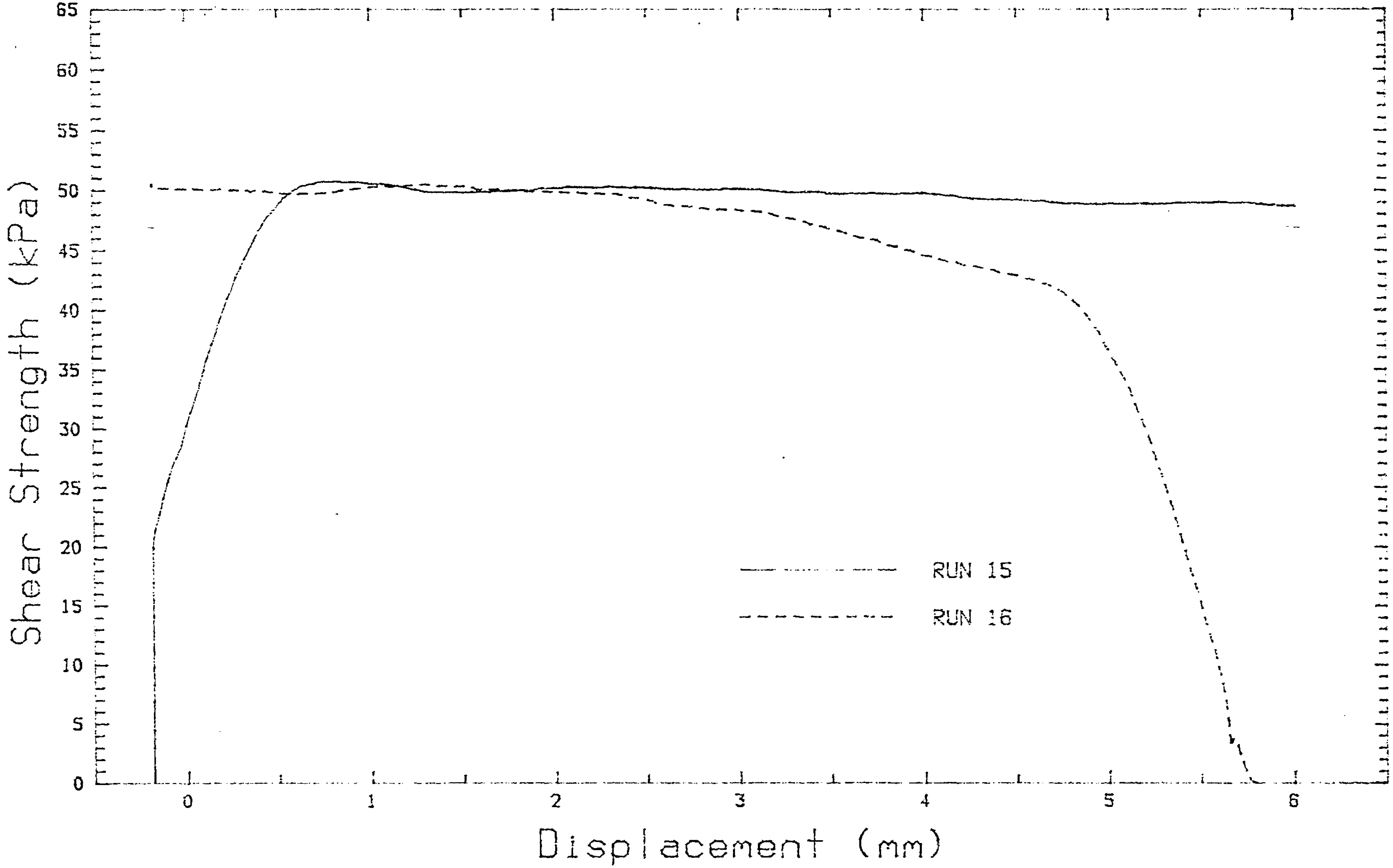

DIRECT SHEAR TEST #1 Rate 0.9mm/min Run 1,3,5,7,9
Shear Strength (kPa)
Displacement (mm)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
0
1
2
3
4
5
6
RUN 1
RUN 3
RUN 5
RUN 7
RUN 9

DIRECT SHEAR TEST #1 Rate 0.9mm/min Run 2,4,6,8,10




DIRECT SHEAR TEST #1 Rate 0.9mm/min Run 11,12,13,14
Shear Strength (kPa)
Displacement (mm)
RUN 11
RUN 12
RUN 13
RUN 14
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0
1
2
3
4
5
6

DIRECT SHEAR TEST #1 Rate 0.12 mm/min Run 15 & 16



## DIRECT SHEAR TEST # 2

Soil : Temuka Clay

Batch : 5

Date of Test : 3rd February 1990

Normal Stress : 200 kPa

Overconsolidation Ratio : 3

Initial Sample Depth : 8.425 mm

### Consolidation Stages

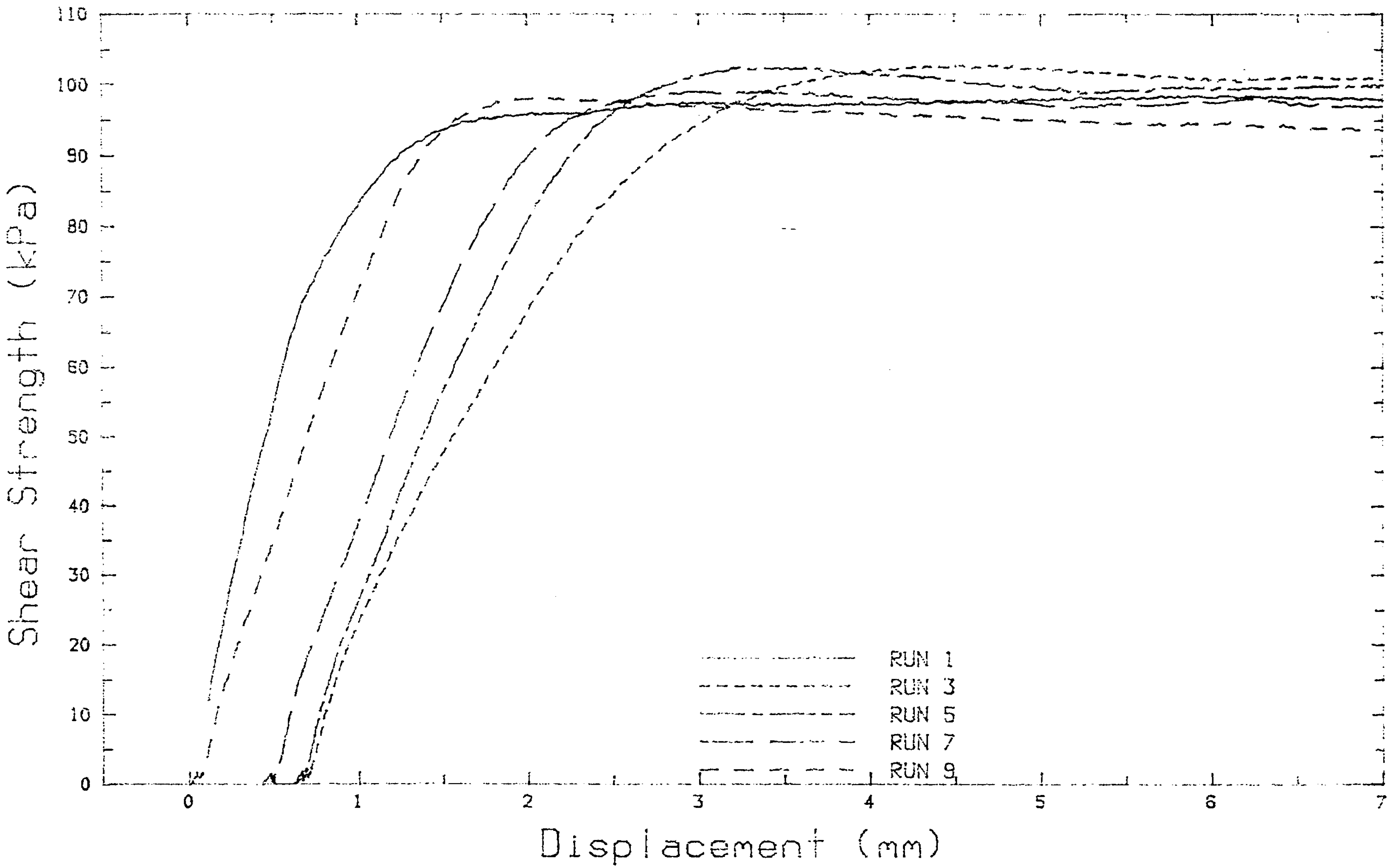
| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 21 kPa | 8.282 mm |
| 2 | 48 kPa | 7.995 mm |
| 3 | 95 kPa | 7.605 mm |
| 4 | 188 kPa | 7.143 mm |
| 5 | 374 kPa | 6.802 mm |
| 6 | 600 kPa | 6.555 mm |
| 7 | 200 kPa | 6.657 mm |

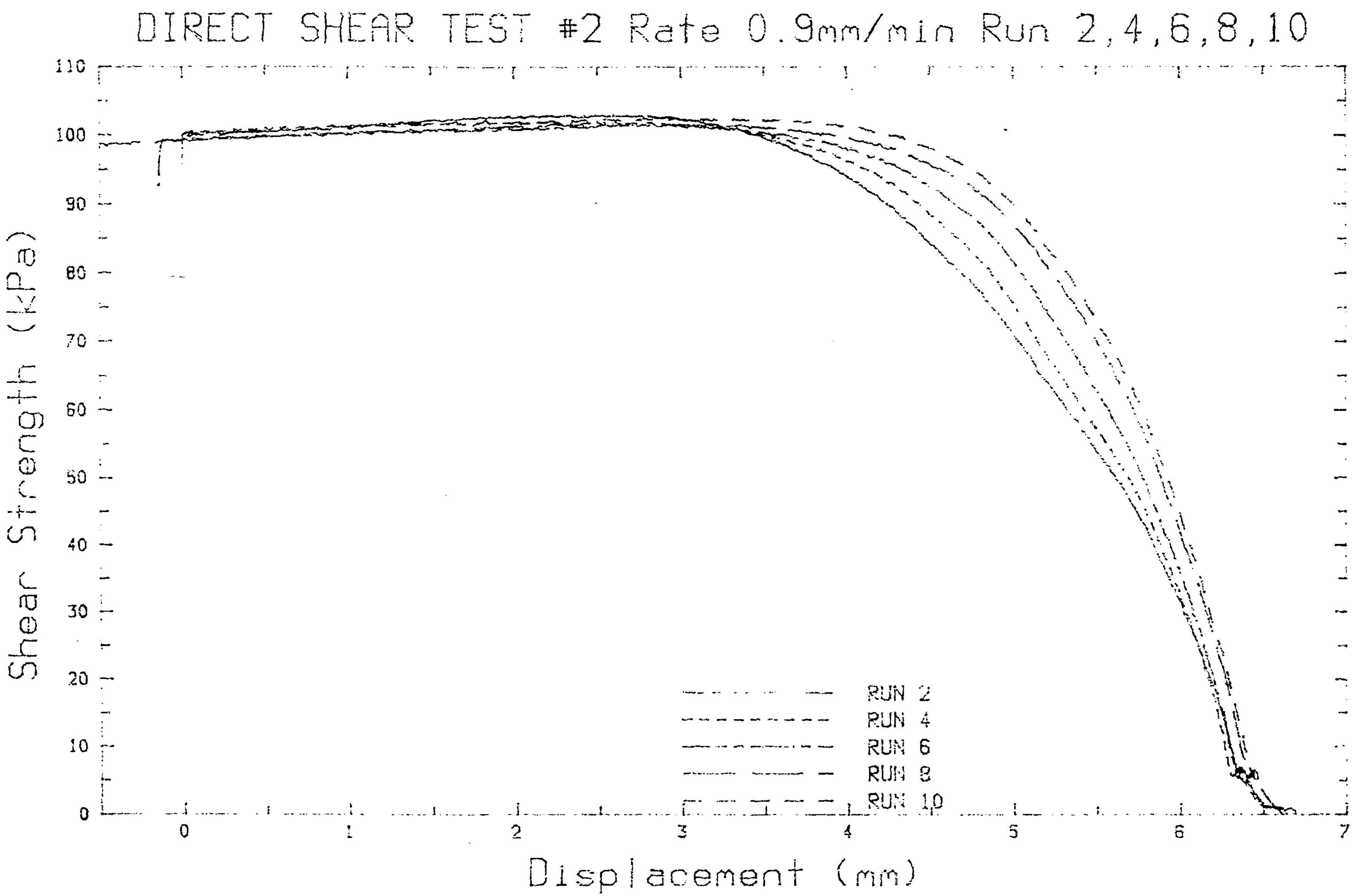
### Shearing Stages

| STAGE | SHEARING RATES |
|---|---|
| 1 - 14 | 0.9 mm/min |
| 15 - 16 | 0.12 mm/min |

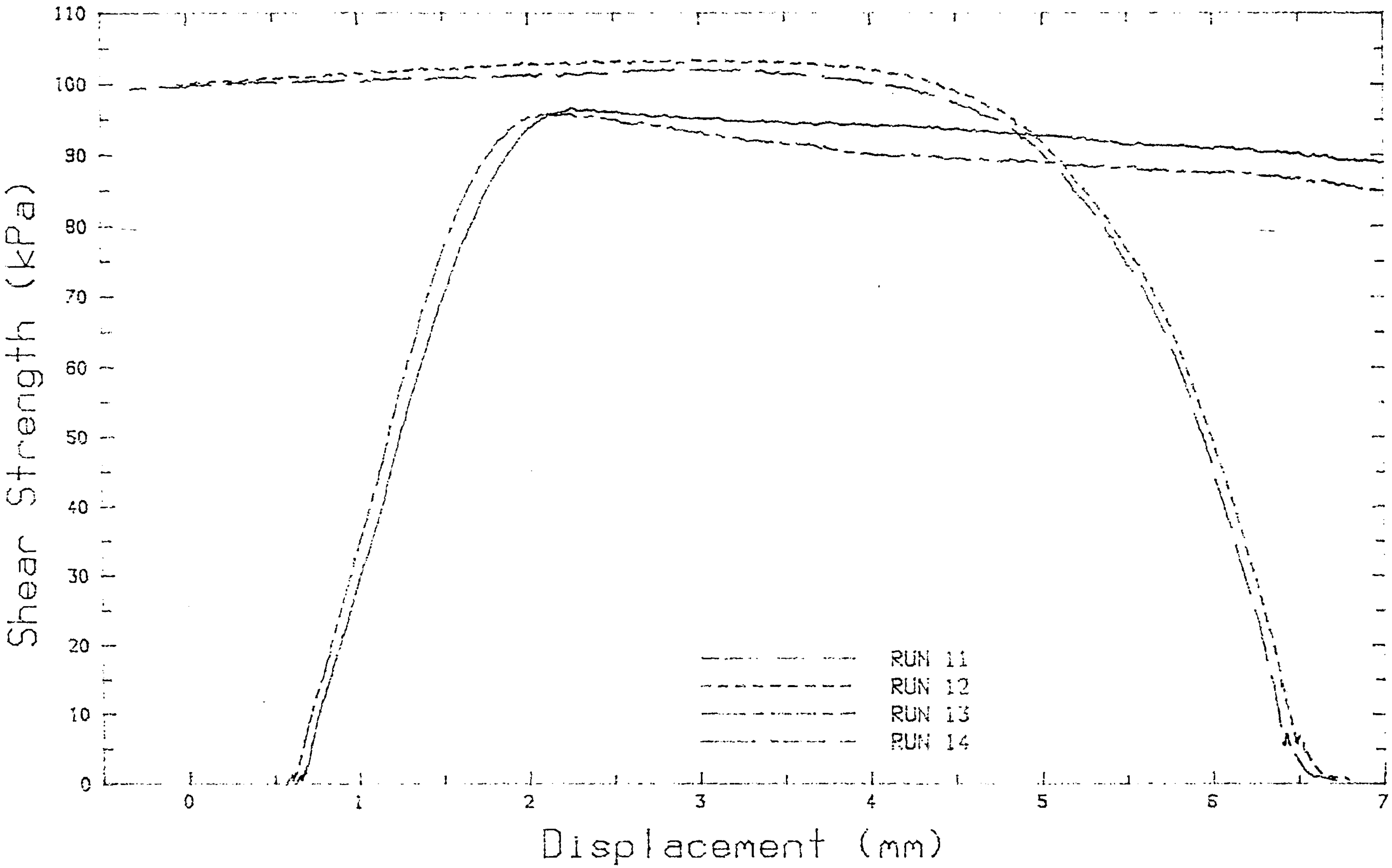
Final Moisture Content : 23.8 %

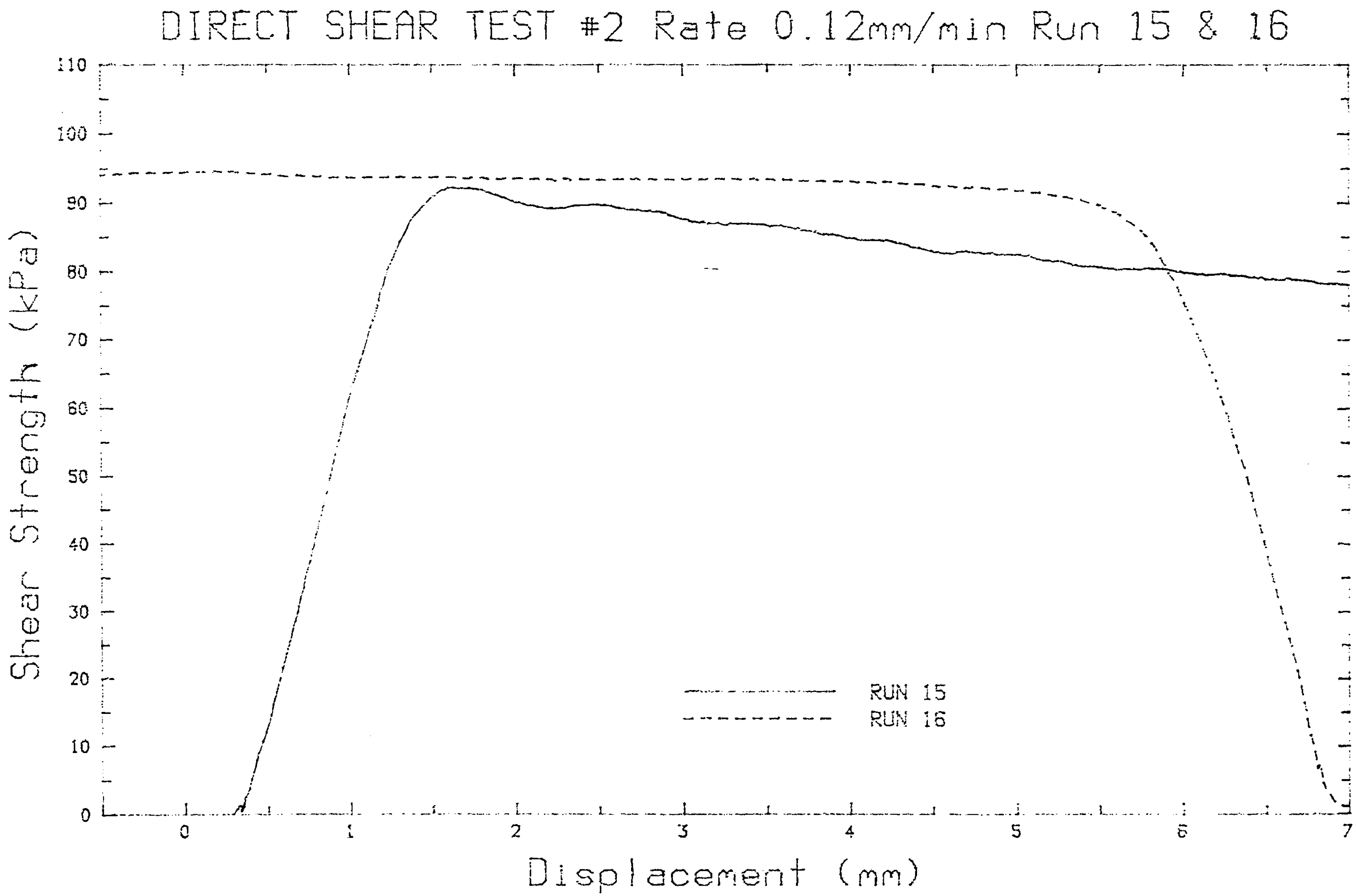
DIRECT SHEAR TEST #2 Rate 0.9mm/min Run 1,3,5,7,9




DIRECT SHEAR TEST #2 Rate 0.9mm/min Run 2,4,6,8,10
Shear Strength (kPa)
Displacement (mm)
RUN 2
RUN 4
RUN 6
RUN 8
RUN 10

DIRECT SHEAR TEST #2 Rate 0.9mm/min Run 11,12,13,14





DIRECT SHEAR TEST #2 Rate 0.12mm/min Run 15 & 16
Shear Strength (kPa)
Displacement (mm)
RUN 15
RUN 16
110
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
5
6
7

DIRECT SHEAR TEST # 3

Soil : Temuka Clay

Batch : 5

Date of Test : 4th February 1990

Normal Stress : 400 kPa

Overconsolidation Ratio : 3

Initial Sample Depth : 8.112 mm

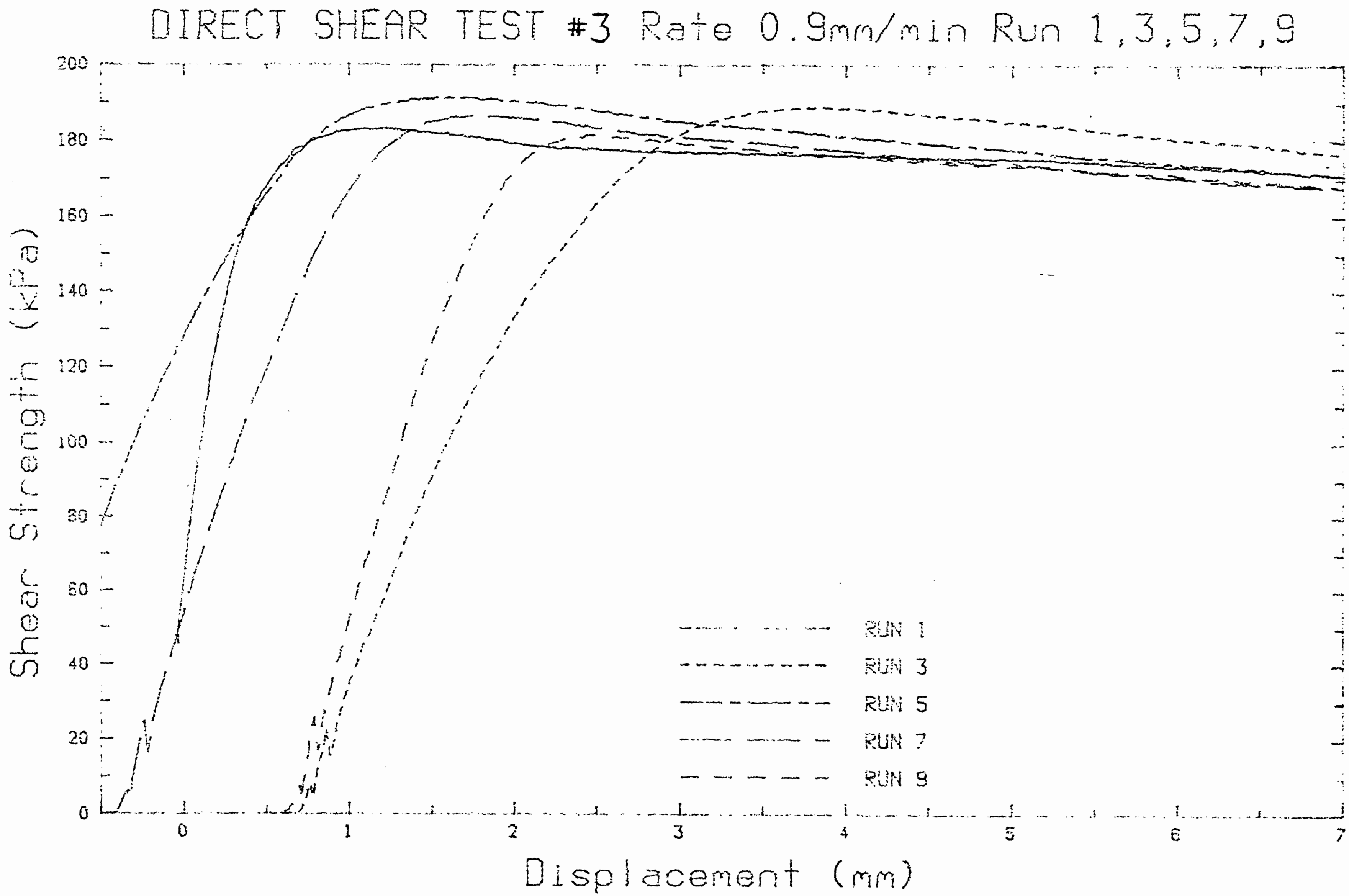
Consolidation Stages

| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 84 kPa | 7.830 mm |
| 2 | 166 kPa | 7.452 mm |
| 3 | 331 kPa | 7.042 mm |
| 4 | 660 kPa | 6.634 mm |
| 5 | 1200 kPa | 6.202 mm |
| 6 | 400 kPa | 6.407 mm |

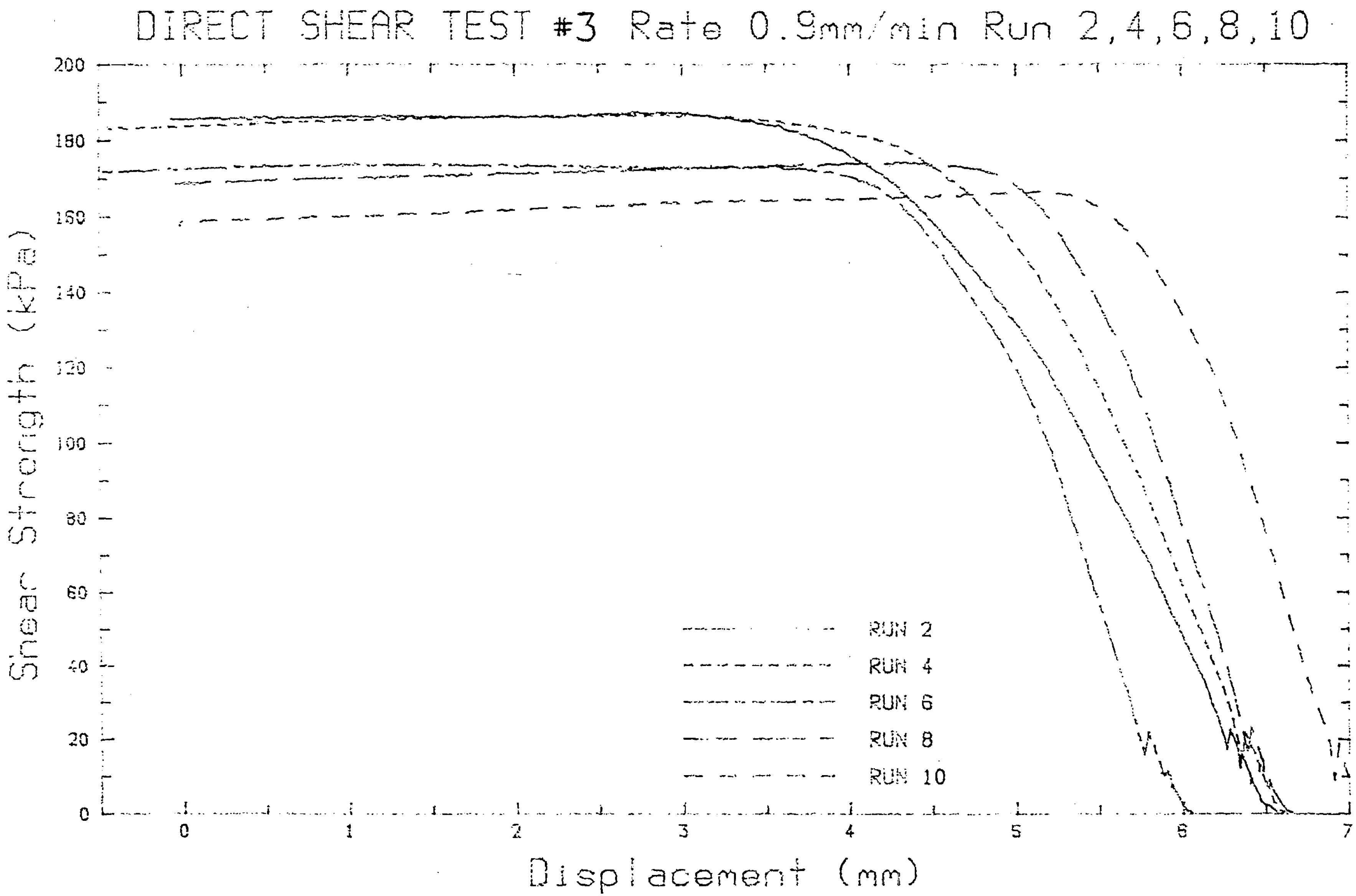
Shearing Stages

| STAGE | SHEARING RATES |
|---|---|
| 1 - 14 | 0.9 mm/min |
| 15 - 16 | 0.12 mm/min |

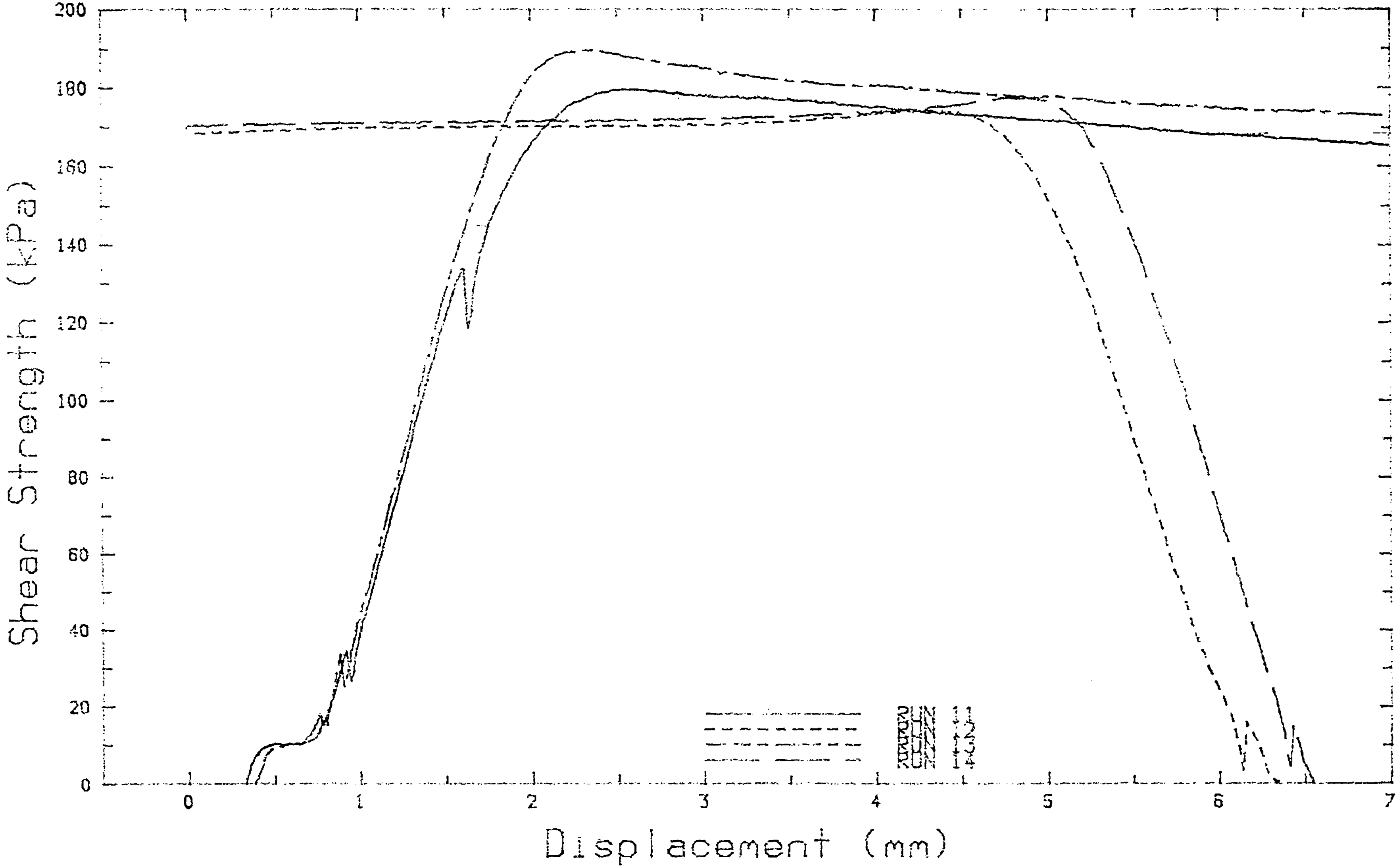
Final Moisture Content : 22.0 %



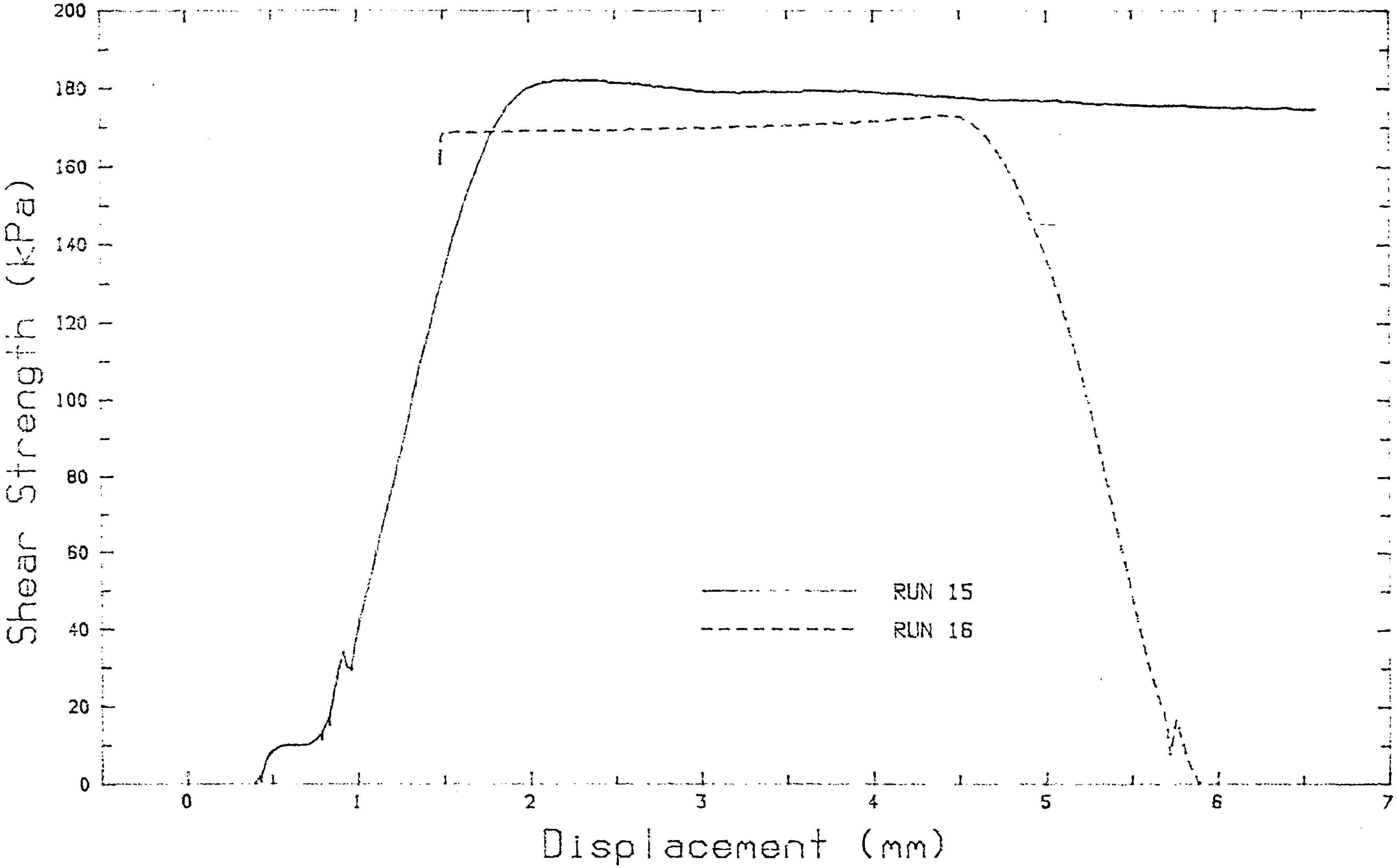
DIRECT SHEAR TEST #3 Rate 0.9mm/min Run 1,3,5,7,9
Shear Strength (kPa)
0
20
40
60
80
100
120
140
160
180
200
Displacement (mm)
0
1
2
3
4
5
6
7
RUN 1
RUN 3
RUN 5
RUN 7
RUN 9


DIRECT SHEAR TEST #3 Rate 0.9mm/min Run 2,4,6,8,10
Shear Strength (kPa)
200
180
160
140
120
100
80
60
40
20
0
0
1
2
3
4
5
6
7
Displacement (mm)
RUN 2
RUN 4
RUN 6
RUN 8
RUN 10

DIRECT SHEAR TEST #3 Rate 0.9mm/min Run 11,12,13,14



DIRECT SHEAR TEST #3 Rate 0.12mm/min Run 15,16



# APPENDIX IV

# RING SHEAR TESTS

SUMMARY OF FIGURES AND TABLES

RING SHEAR TEST # 1

Soil : Temuka Clay

Batch : 4

Date of Test : 25th May 1989

Normal Stress : 100 kPa

Overconsolidation Ratio : 3

Initial Moisture Content : 28.4 %

Initial Sample Depth : 5.910 mm

Consolidation Stages

| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 16 kPa | 5.730 mm |
| 2 | 42 kPa | 5.370 mm |
| 3 | 94 kPa | 5.029 mm |
| 4 | 200 kPa | 4.665 mm |
| 5 | 300 kPa | 4.450 mm |
| 6 | 100 kPa | 4.539 mm |

Shearing Stages

| STAGE | SHEARING RATES | SAMPLE DEPTH |
|---|---|---|
| 1 - 8 | #7 | 1.624 mm |

Final Moisture Content : 29.3 %

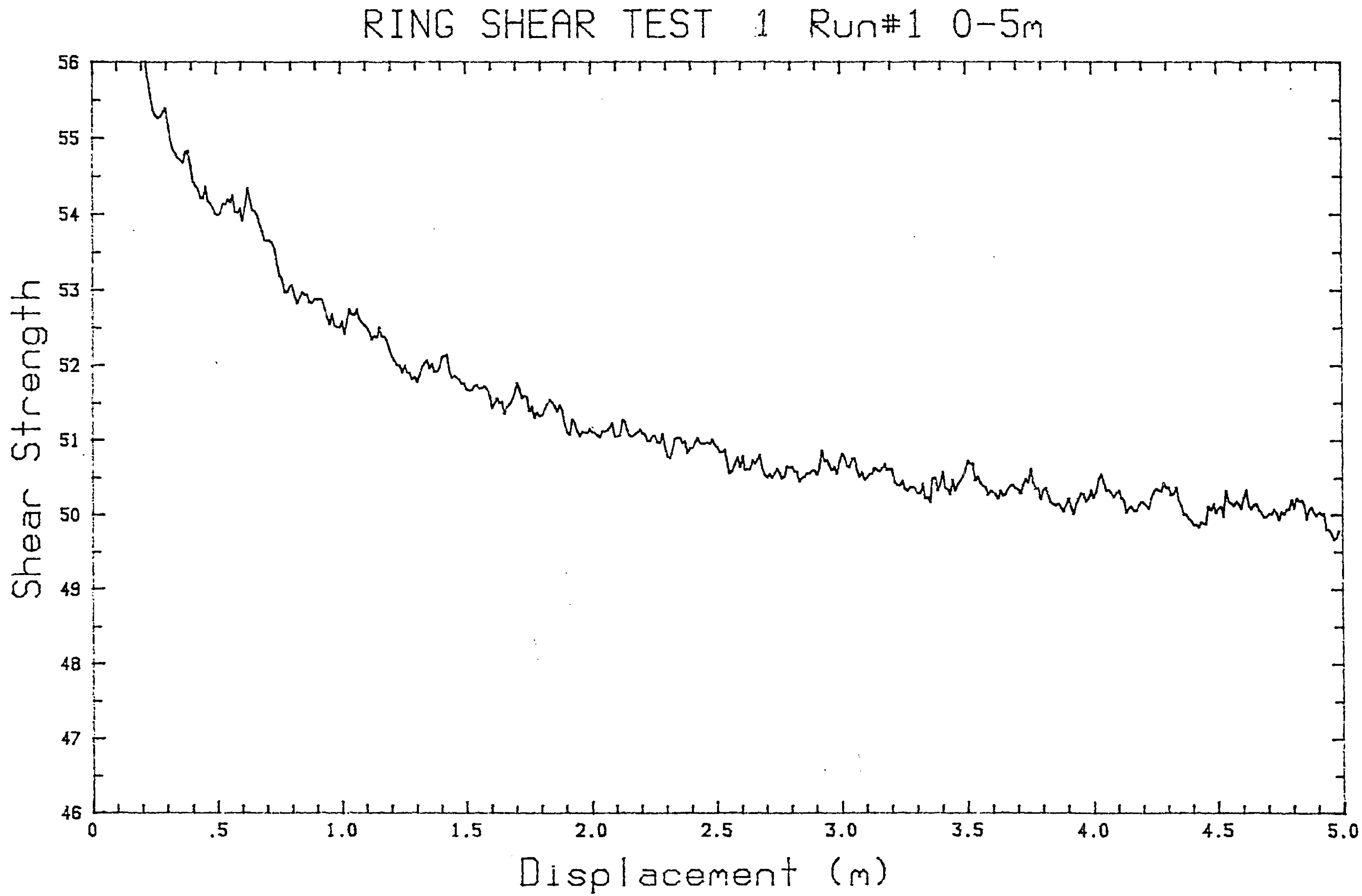
Table A4.1a TEST #1 PROGRAMME

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 0:00 | Rate 7 ON | 0.000 | ------- | ----------- | -------- |
| | | | 1 | #7 46 | 108 |
| 1:48 | Rate 7 ON | 5.000 | ------- | ----------- | -------- |
| | | | 2 | #7 46 | 108 |
| 3:36 | Rate 7 ON | 10.000 | ------- | ----------- | -------- |
| | | | 3 | #7 46 | 108 |
| 5:24 | Rate 7 ON | 15.000 | ------- | ----------- | -------- |
| | | | 4 | #7 46 | 108 |
| 7:12 | Rate 7 ON | 20.000 | ------- | ----------- | -------- |
| | | | 5 | #7 46 | 115 |
| 9:06* | Rate 7 ON | 25.300 | ------- | ----------- | -------- |
| | | | 6 | #7 46 | 154 |
| 11:44 | Rate 7 ON | 32.400 | ------- | ----------- | -------- |
| | | | 7 | #7 46 | 161 |
| 14:25 | Rate 7 ON | 39.800 | ------- | ----------- | -------- |
| | | | 8 | #7 46 | 722 |
| 26:27 | Rate 7 ON | 73.000 | ------- | ----------- | -------- |

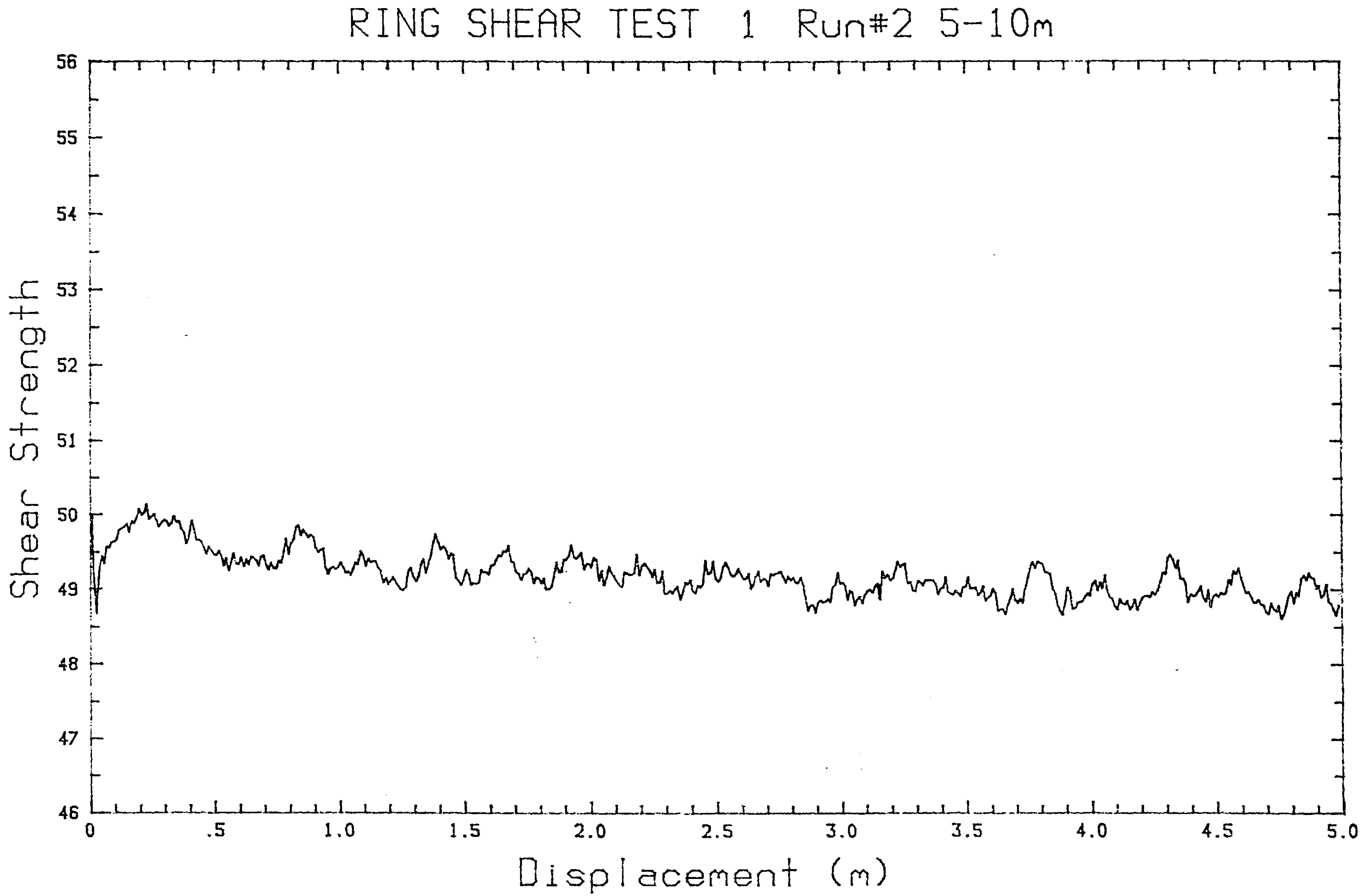
* Floppy Disk Error - 1.9m data lost.

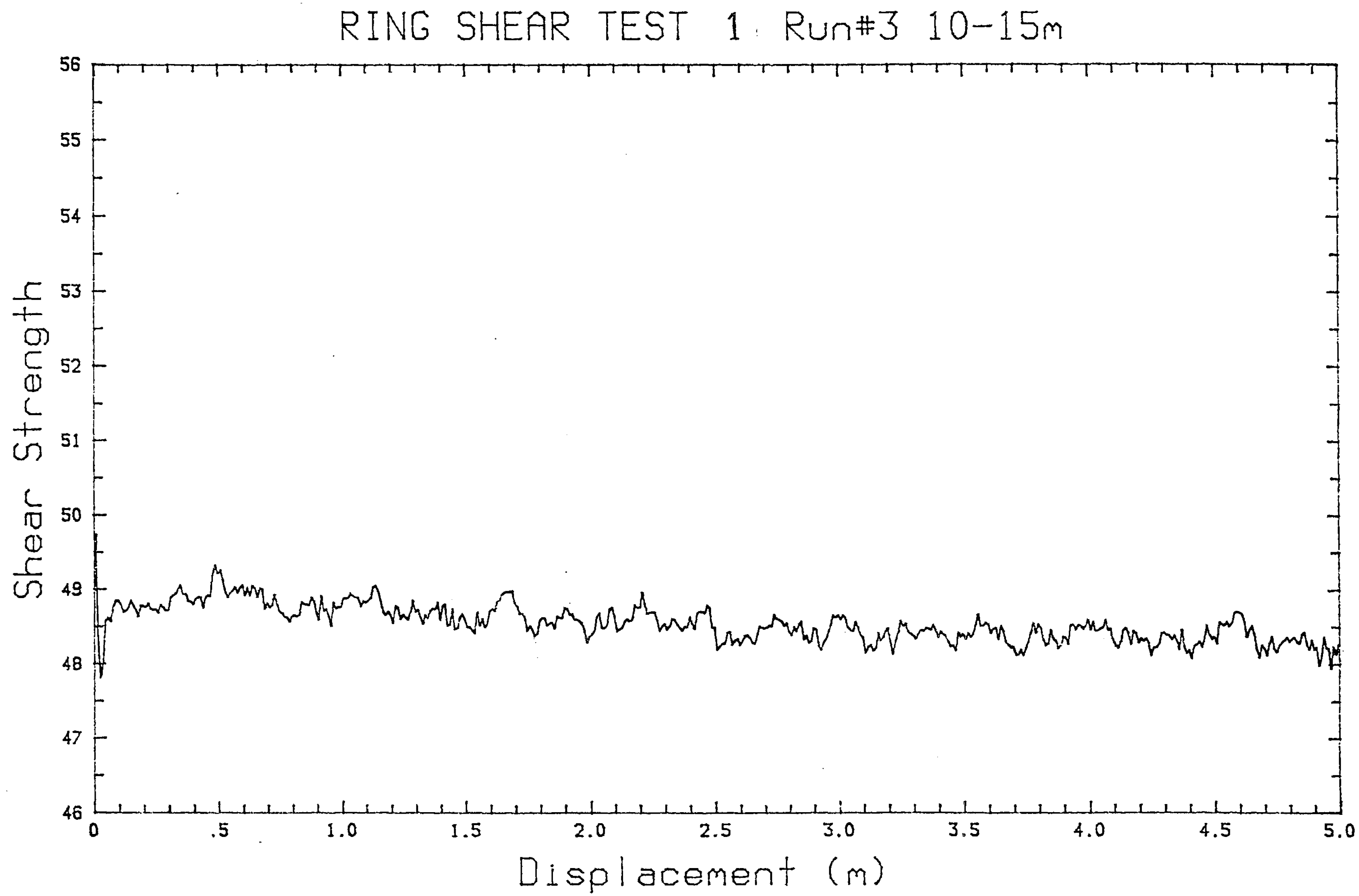
Table A4.1b TEST #1 SAMPLE DEGRADATION AND WATER DEMAND

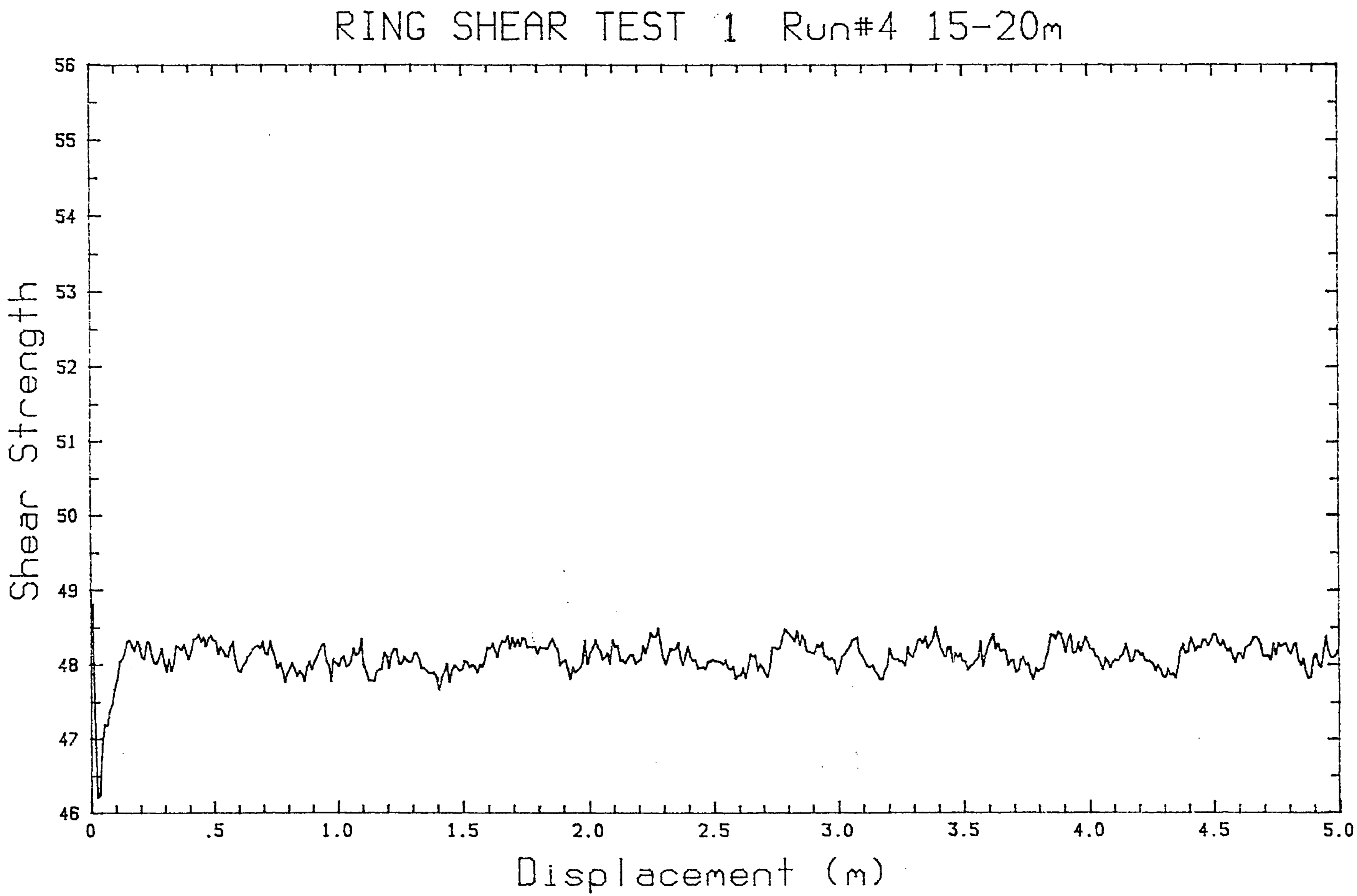
| Run No. | Shear Rate (mm/min) | Duration (min) | Δ Depth (mm) | Water Demand (ml) |
|---|---|---|---|---|
| 1 | #7 46.0 | 108 | 0.575 | 0.51 |
| 2 | #7 46.0 | 108 | 0.410 | 0.26 |
| 3 | #7 46.0 | 108 | 0.364 | 0.21 |
| 4 | #7 46.0 | 108 | 0.260 | 0.18 |
| 5 | #7 46.0 | 115 | 0.249 | 0.16 |
| 6 | #7 46.0 | 154 | 0.209 | 0.15 |
| 7 | #7 46.0 | 161 | 0.096 | 0.07 |
| 8 | #7 46.0 | 722 | 0.755 | 0.38 |



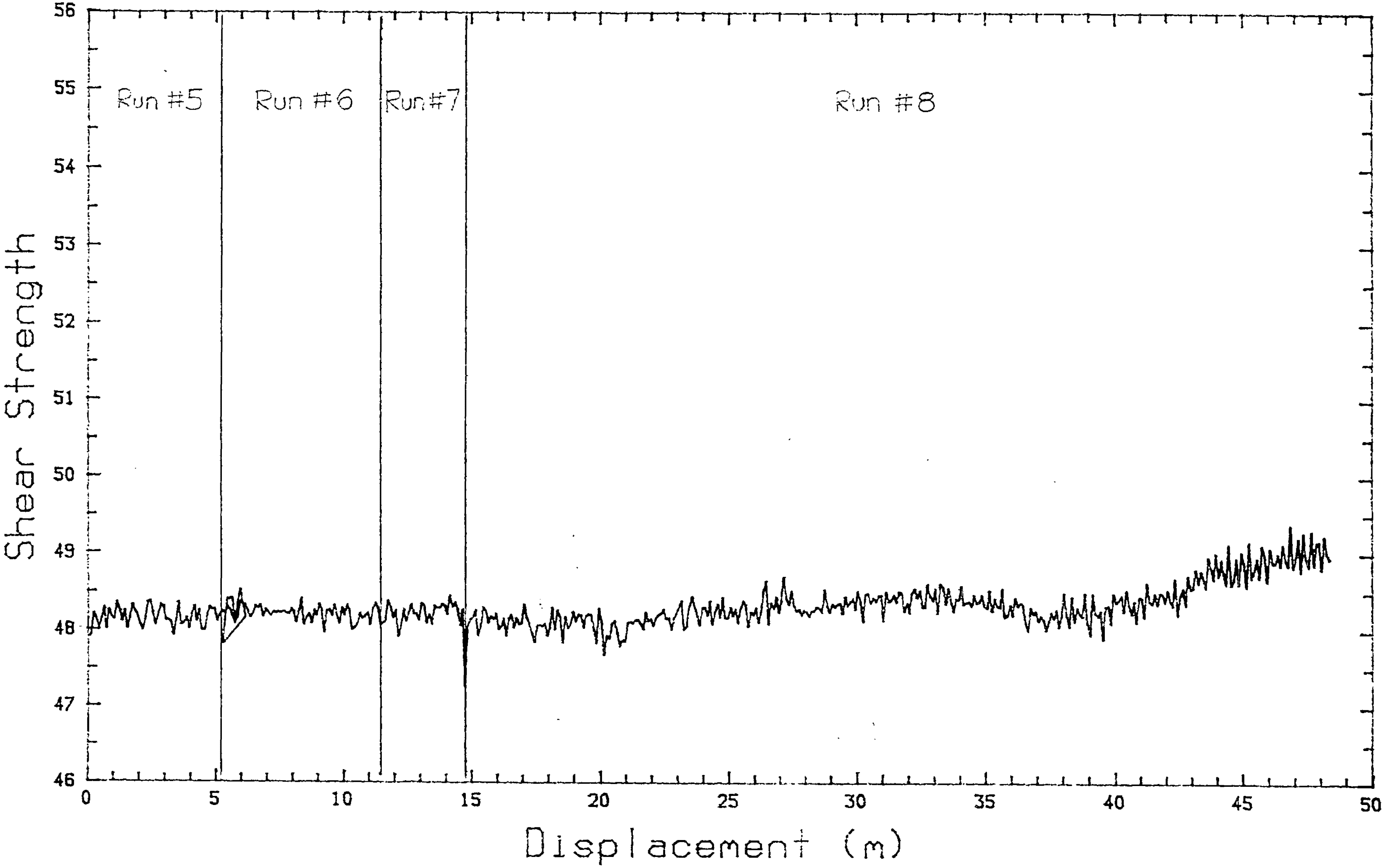
RING SHEAR TEST 1 Run#1 0-5m
Shear Strength
56
55
54
53
52
51
50
49
48
47
46
0
.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Displacement (m)

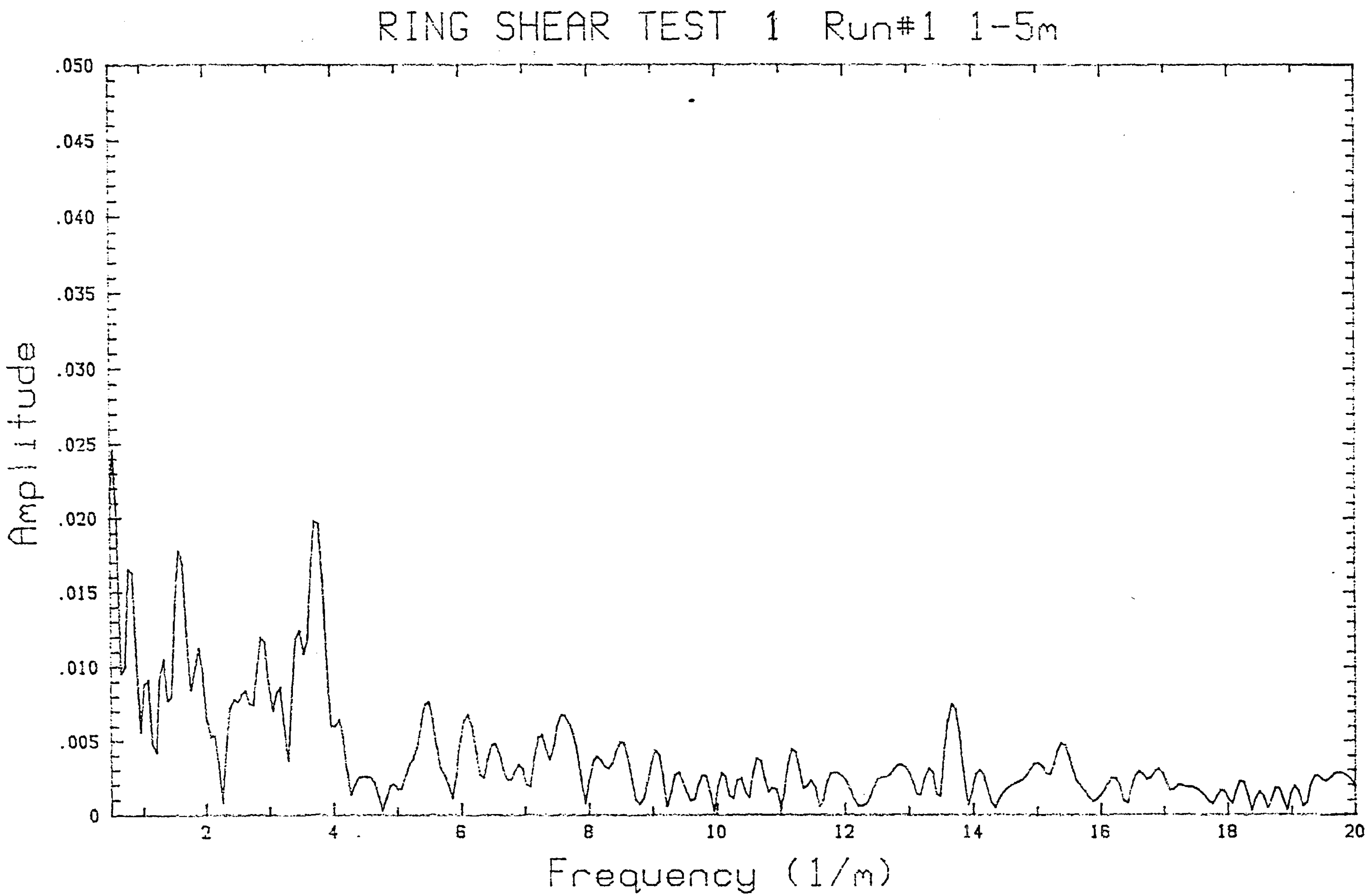

RING SHEAR TEST 1 Run#2 5-10m
Shear Strength
Displacement (m)
56
55
54
53
52
51
50
49
48
47
46
0
.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0

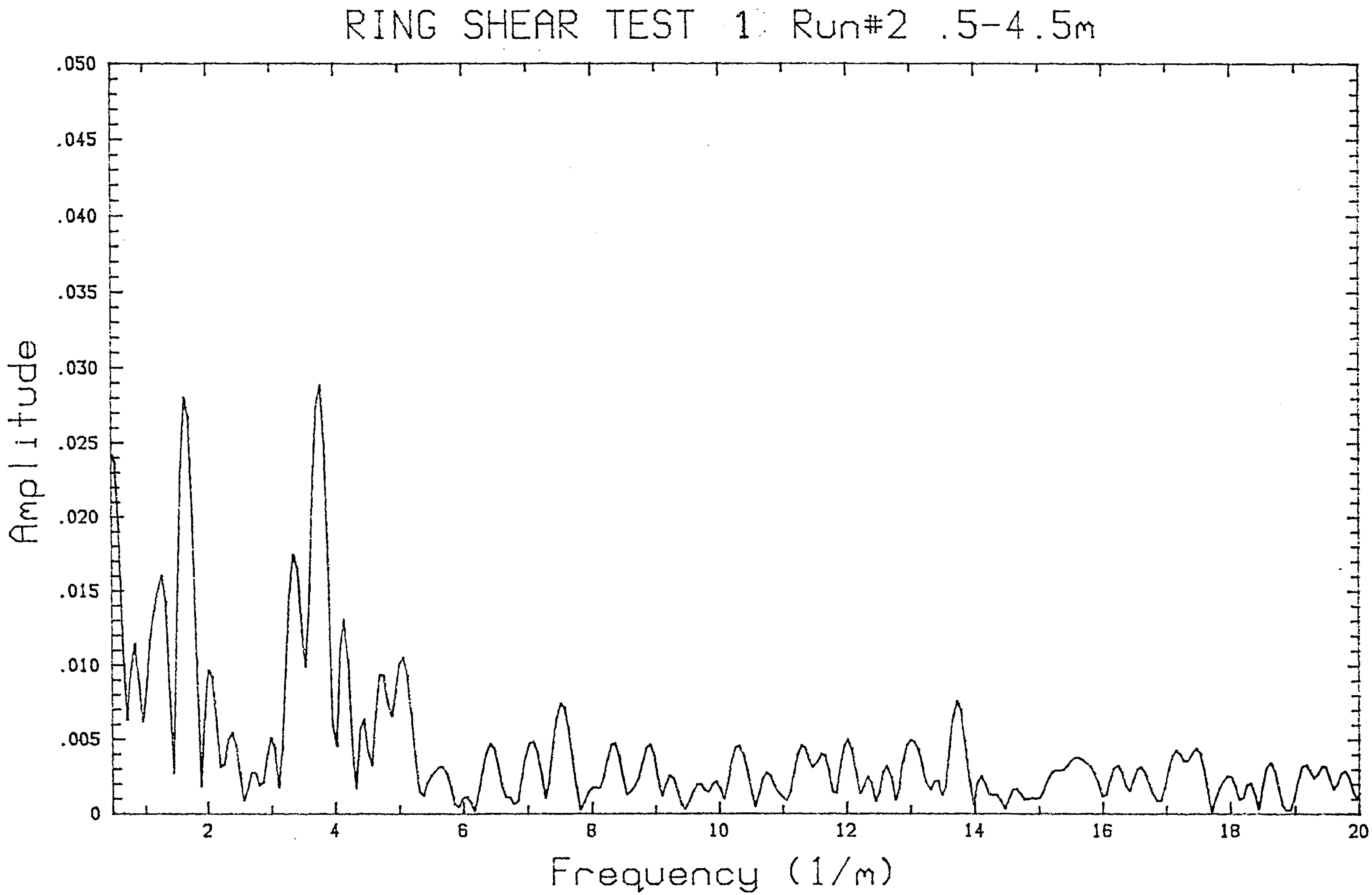

RING SHEAR TEST 1 Run#3 10-15m
Shear Strength
56
55
54
53
52
51
50
49
48
47
46
0
.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Displacement (m)


RING SHEAR TEST 1 Run#4 15-20m
Shear Strength
56
55
54
53
52
51
50
49
48
47
46
0
.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Displacement (m)

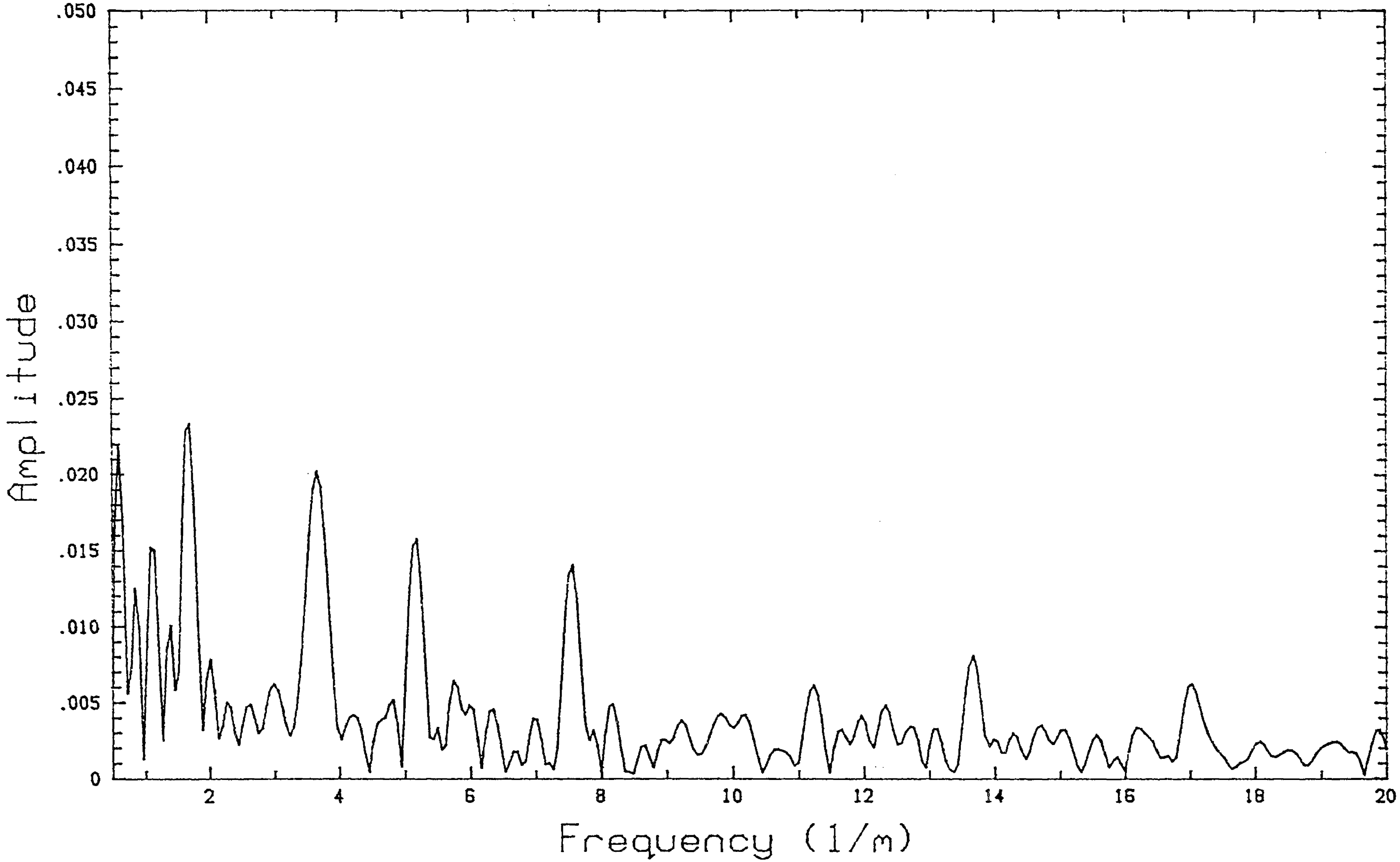
RING SHEAR TEST 1 Run#5-8 20-70m



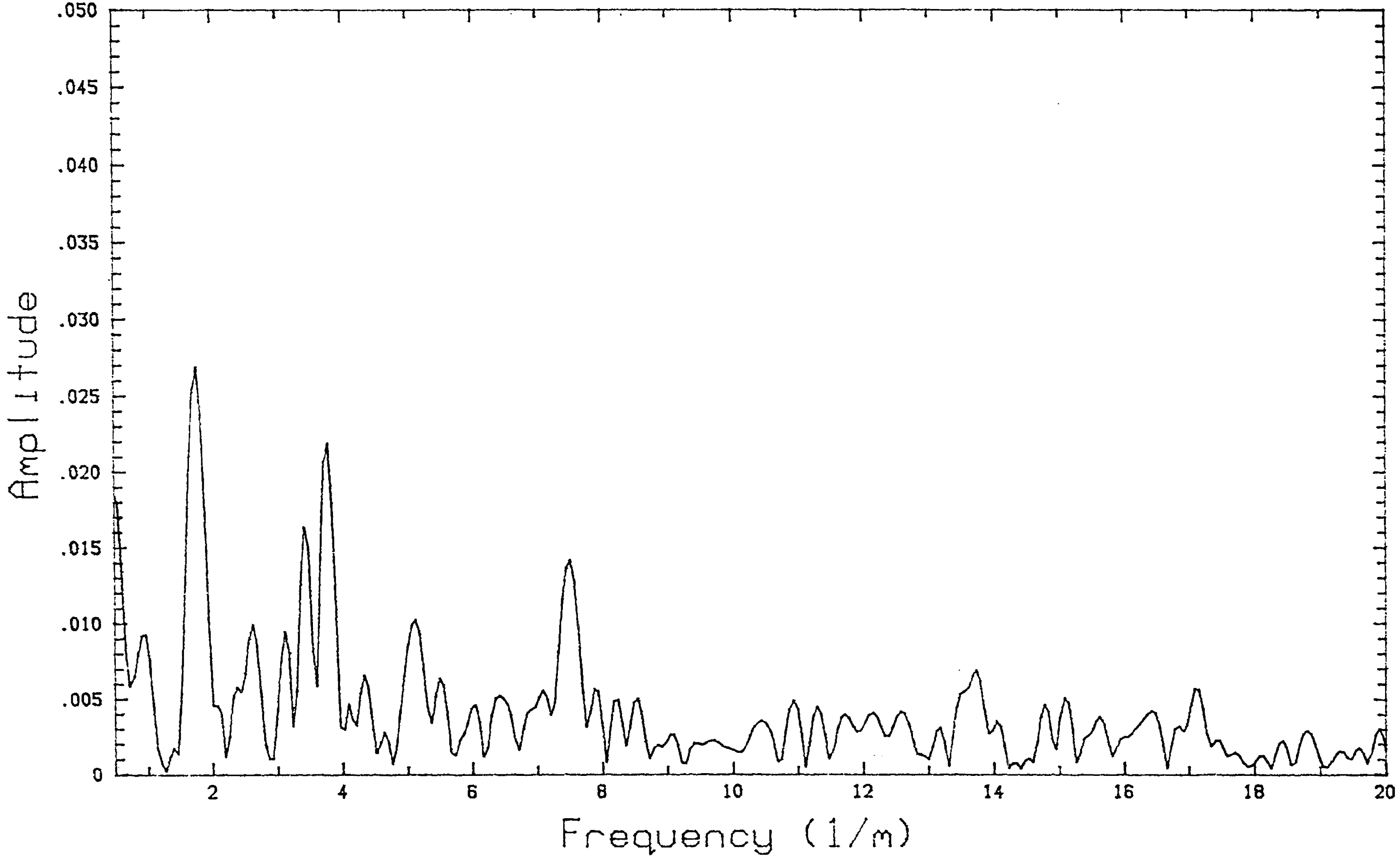

RING SHEAR TEST 1 Run#1 1-5m
Amplitude
Frequency (1/m)
.050
.045
.040
.035
.030
.025
.020
.015
.010
.005
0
2
4
6
8
10
12
14
16
18
20


RING SHEAR TEST 1 Run#2 .5-4.5m
.050
.045
.040
.035
.030
.025
.020
.015
.010
.005
0
Amplitude
2
4
6
8
10
12
14
16
18
20
Frequency (1/m)

RING SHEAR TEST 1 Run#3 10-15m



RING SHEAR TEST 1 Run#4 15-20m



RING SHEAR TEST #2

Soil : Temuka Clay

Batch : 5

Date of Test : 28th September 1989

Normal Stress : 100 kPa

Overconsolidation Ratio : 3

Initial Moisture Content : 29.0 %

Initial Sample Depth : 7.182 mm

Consolidation Stages

| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 16 kPa | 6.601 mm |
| 2 | 42 kPa | 6.292 mm |
| 3 | 94 kPa | 5.965 mm |
| 4 | 200 kPa | 5.602 mm |
| 5 | 300 kPa | 5.396 mm |
| 6 | 100 kPa | 5.514 mm |

Shearing Stages

| STAGE | SHEARING RATES | SAMPLE DEPTH |
|---|---|---|
| 1 | #7 | 2.705 mm |
| 2 | #4, #5, #6 | 2.512 mm |
| 3 | #1, #2, #3, #4 | 2.510 mm |
| 4 | #4 | 2.503 mm |
| 5 | #8, #9 | 1.334 mm |

Final Moisture Content : 30.1 %

Table A4.2a TEST #2 PROGRAMME

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 0:00 | Rate 7 ON | 0.000 | ------- | ----------- | -------- |
| 1:48 | Rate 4t FOR | | | | |
| 1:59 | Rate 4t REV | | | | |
| 2:20 | Rate 4t FOR | | 1 | #7 46 | 220 |
| 3:03 | Rate 4t REV | | | | |
| 3:40 | | 10.120 | ------- | ----------- | -------- |
| 3:56 | Rate 4t FOR | | 2.1 | #7 46 | 17 |
| 3:57 | Rate 7 OFF | 10.902 | ------- | ----------- | -------- |
| | | | 2.2 | #4 .124 | 41 |
| 4:38 | Rate 6 ON | 10.907 | ------- | ----------- | -------- |
| 4:39 | Rate 4t REV | | | | |
| | | | 2.3 | #6 7.0 | 51 |
| 5:28 | Rate 4t FOR | | | | |
| 5:29 | Rate 6 OFF | 11.264 | ------- | ----------- | -------- |
| | | | 2.4 | #4 .124 | 54 |
| 6:23 | Rate 5 ON | 11.271 | ------- | ----------- | -------- |
| 6:24 | Rate 4t REV | | | | |
| | | | 2.5 | #5 0.93 | 53 |
| 7:14 | Rate 4t FOR | | | | |
| 7:16 | Rate 5 OFF | 11.320 | ------- | ----------- | -------- |
| | | | 2.6 | #4 .124 | 25 |
| 7:41 | Rate 4p ON | 11.323 | ------- | ----------- | -------- |
| 7:42 | Rate 4t OFF | | | | |
| 7:43 | Rate 3 REV | | 3.1 | #4 .124 | 20 |
| 7:59 | Rate 3 FOR | | | | |
| 8:01 | Rate 4p OFF | 11.326 | ------- | ----------- | -------- |
| | | | 3.2 | #3 .018 | 63 |
| 9:04 | Rate 4p ON | 11.327 | ------- | ----------- | -------- |
| 9:04 | Rate 3 REV | | | | |
| | | | 3.3 | #4 .124 | 40 |
| 9:42 | Rate 3 OFF | | | | |
| 9:43 | Rate 2 FOR | | | | |
| 9:44 | Rate 4p OFF | 11.332 | ------- | ----------- | -------- |
| | | | 3.4 | #2 .0022 | 201 |
| 13:05 | Rate 4p ON | 11.332 | ------- | ----------- | -------- |
| 13:06 | Rate 2 OFF | | | | |
| | | | 3.5 | #4 .124 | 41 |
| 13:07 | Rate 1 FOR | | | | |
| 13:46 | Rate 4p OFF | 11.337 | ------- | ----------- | -------- |
| | | | 3.6 | #1 .00029 | 609 |
| 23:55 | Rate 4p ON | 11.337 | ------- | ----------- | -------- |
| 23:56 | Rate 1 OFF | | | | |
| 23:57 | Rate 3 REV | | 3.7 | #4 .124 | 40 |

Table A4.2a TEST #2 PROGRAMME (continued)

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 24:35 | | 11.342 | | | |
| 25:57 | Rate 3 OFF | | 4.1 | #4 .124 | 1210 |
| 44:45 | | 11.492 | | | |
| | | | 5.1 | #4 .124 | 8 |
| 44:53 | Rate 4t FOR | 11.493 | | | |
| 44:54 | Rate 4p OFF | | 5.2 | #4 .124 | 15 |
| 45:08 | Rate 4t REV* | | | | |
| 45:11 | Rate 8 ON | 11.495 | | | |
| | | | 5.3 | #8 310 | 18 |
| 45:28 | Rate 4t FOR | | | | |
| 45:29 | Rate 8 OFF | 17.075 | | | |
| | | | 5.4 | #4 .124 | 19 |
| 45:48 | Rate 9 ON | 17.077 | | | |
| 45:49 | Rate 4t REV | | | | |
| | | | 5.5 | #9 2079 | 22 |
| 46:09 | Rate 4t FOR | | | | |
| 46:10 | Rate 9 OFF | 62.815 | | | |
| 46:35 | Rate 4p ON | 62.818 | 5.6 | #4 .124 | 25 |
| 46:36 | Rate 4t OFF | | | | |

* Error made in direction of secondary shear drive resulting in drop in strength and rescheduling of rates.

Table A4.2b TEST #2 SAMPLE DEGRADATION AND WATER DEMAND

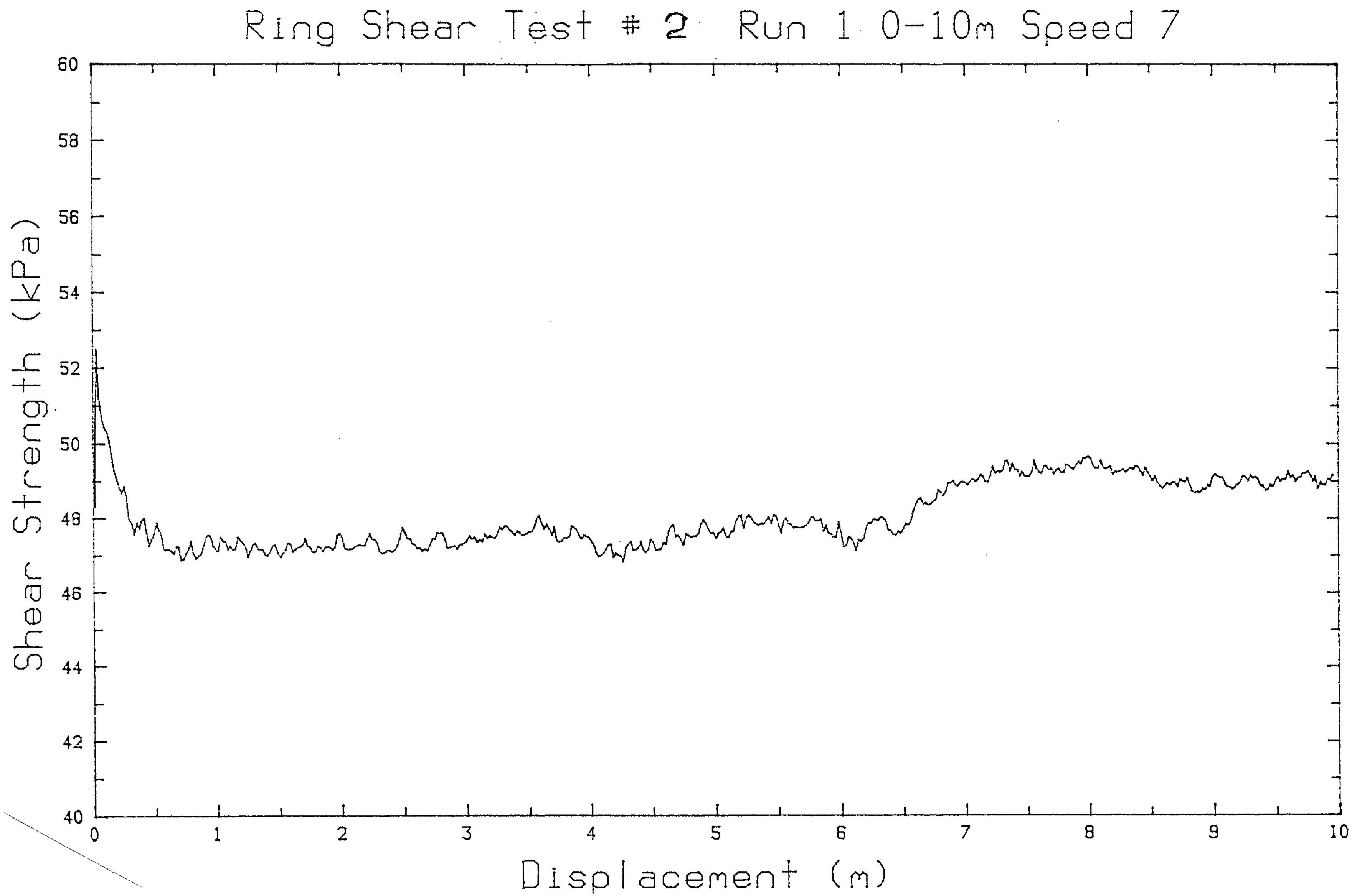
| Run No. | Shear Rate (mm/min) | Duration (min) | Δ Depth (mm) | Water Demand (ml) |
|---|---|---|---|---|
| 1.0 | #7 46.0 | 220 | 2.859 | 0.26 |
| 2.1 | #7 46.0 | 17 | 0.064 | 0.00 |
| 2.2 | #4t 0.124 | 41 | 0.011 | 0.00 |
| 2.3 | #6 7.0 | 51 | 0.038 | 0.00 |
| 2.4 | #4t 0.124 | 54 | 0.004 | 0.00 |
| 2.5 | #5 0.93 | 53 | 0.009 | 0.00 |
| 2.6 | #4t 0.124 | 25 | 0.000 | 0.00 |
| 3.1 | #4p 0.124 | 20 | 0.004 | 0.00 |
| 3.2 | #3 0.018 | 63 | 0.000 | 0.00 |
| 3.3 | #4p 0.124 | 40 | 0.000 | 0.00 |
| 3.4 | #2 0.0022 | 201 | 0.002 | 0.00 |
| 3.5 | #4p 0.124 | 41 | 0.002 | 0.00 |
| 3.6 | #1 0.00029 | 609 | 0.002 | 0.00 |
| 3.7 | #4p 0.124 | 40 | 0.000 | 0.00 |
| 4.1* | #4p 0.124 | 1210 | 0.006 | 0.00 |
| 5.1 | #4p 0.124 | 8 | 0.008 | 0.01 |
| 5.2 | #4t 0.124 | 15 | 0.002 | 0.01 |
| 5.3 | #8 310 | 18 | 0.165 | 0.17 |
| 5.4 | #4t 0.124 | 19 | 0.028 | 0.02 |
| 5.5 | #9 2079 | 22 | 0.848 | 0.49 |
| 5.6 | #4t 0.124 | 25 | 0.120 | -0.02 |

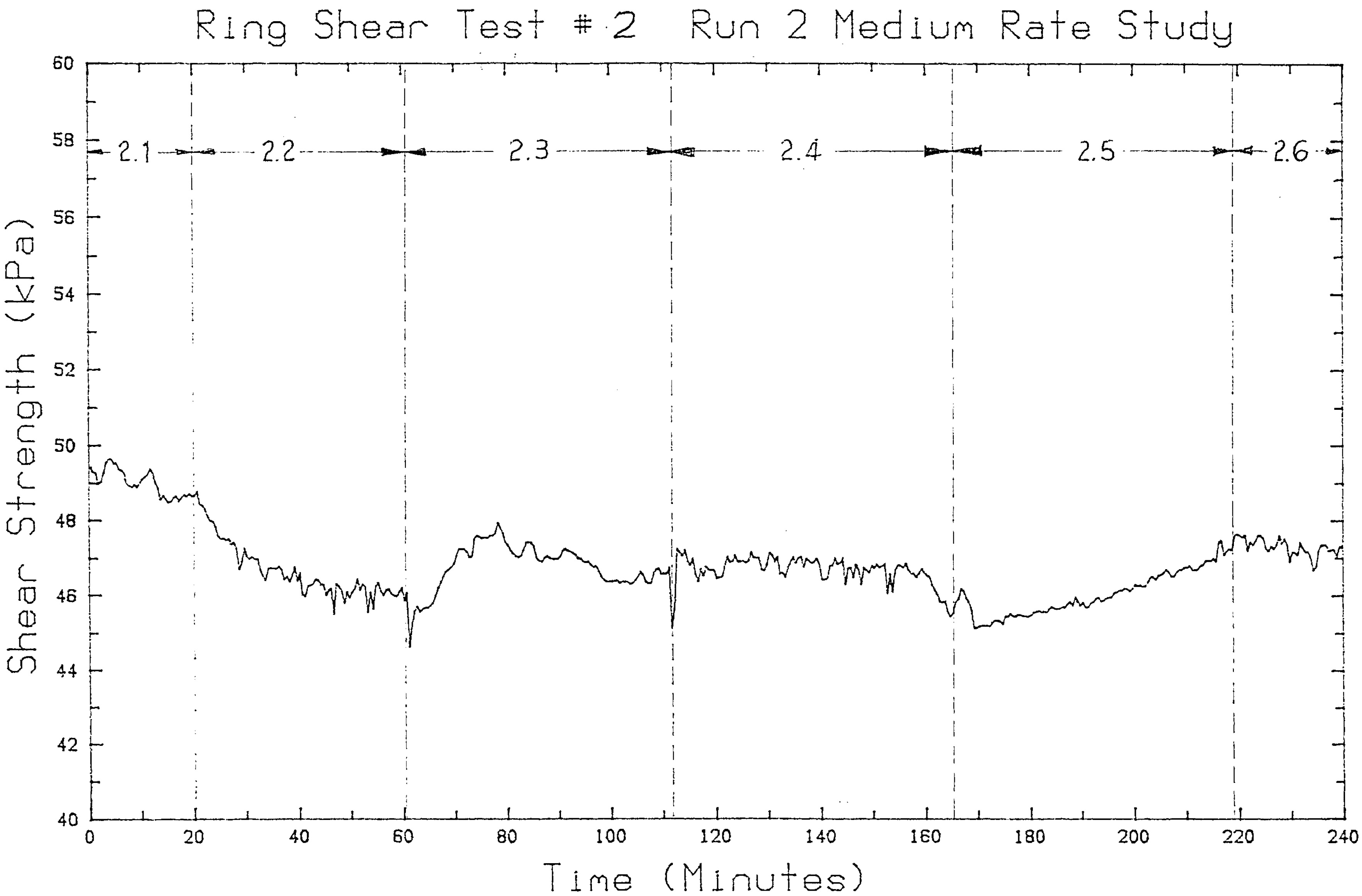
Table A4.2c ANALYSIS OF TEST #2 RESULTS

| Run No. | Shear Rate (mm/min) | Duration (min) | Average $\tau$ (kPa) | Stand. Dev. $\tau$ (kPa) |
|---|---|---|---|---|
| 1.0 | #7 46.0 | 220 | 48.084 | 0.832 |
| 2.1 | #7 46.0 | 17 | 49.013 | 0.384 |
| 2.2 | #4t 0.124 | 41 | 46.660 | 1.115 |
| 2.3 | #6 7.0 | 51 | 46.772 | 0.614 |
| 2.4 | #4t 0.124 | 54 | 46.722 | 0.938 |
| 2.5 | #5 0.93 | 53 | 45.896 | 0.487 |
| 2.6 | #4t 0.124 | 25 | 47.250 | 0.854 |
| 3.1 | #4p 0.124 | 20 | 47.711 | 0.133 |
| 3.2 | #3 0.018 | 63 | 48.107 | 0.693 |
| 3.3 | #4p 0.124 | 40 | 47.232 | 0.162 |
| 3.4 | #2 0.0022 | 201 | 49.210 | 0.826 |
| 3.5 | #4p 0.124 | 41 | 47.562 | 0.226 |
| 3.6 | #1 0.00029 | 609 | 51.340 | 1.379 |
| 3.7 | #4p 0.124 | 40 | 46.774 | 0.755 |
| 4.1* | #4p 0.124 | 1210 | 46.322 | 0.389 |
| 5.1 | #4p 0.124 | 8 | 47.649 | 0.348 |
| 5.2 | #4t 0.124 | 15 | 47.535 + | 0.655 + |
| 5.3 | #8 310 | 18 | 47.918 + | 0.499 + |
| 5.4 | #4t 0.124 | 19 | 45.389 + | 0.747 + |
| 5.5 | #9 2079 | 22 | 54.190 + | 0.533 + |
| 5.6 | #4t 0.124 | 25 | 43.538 + | 0.162 + |

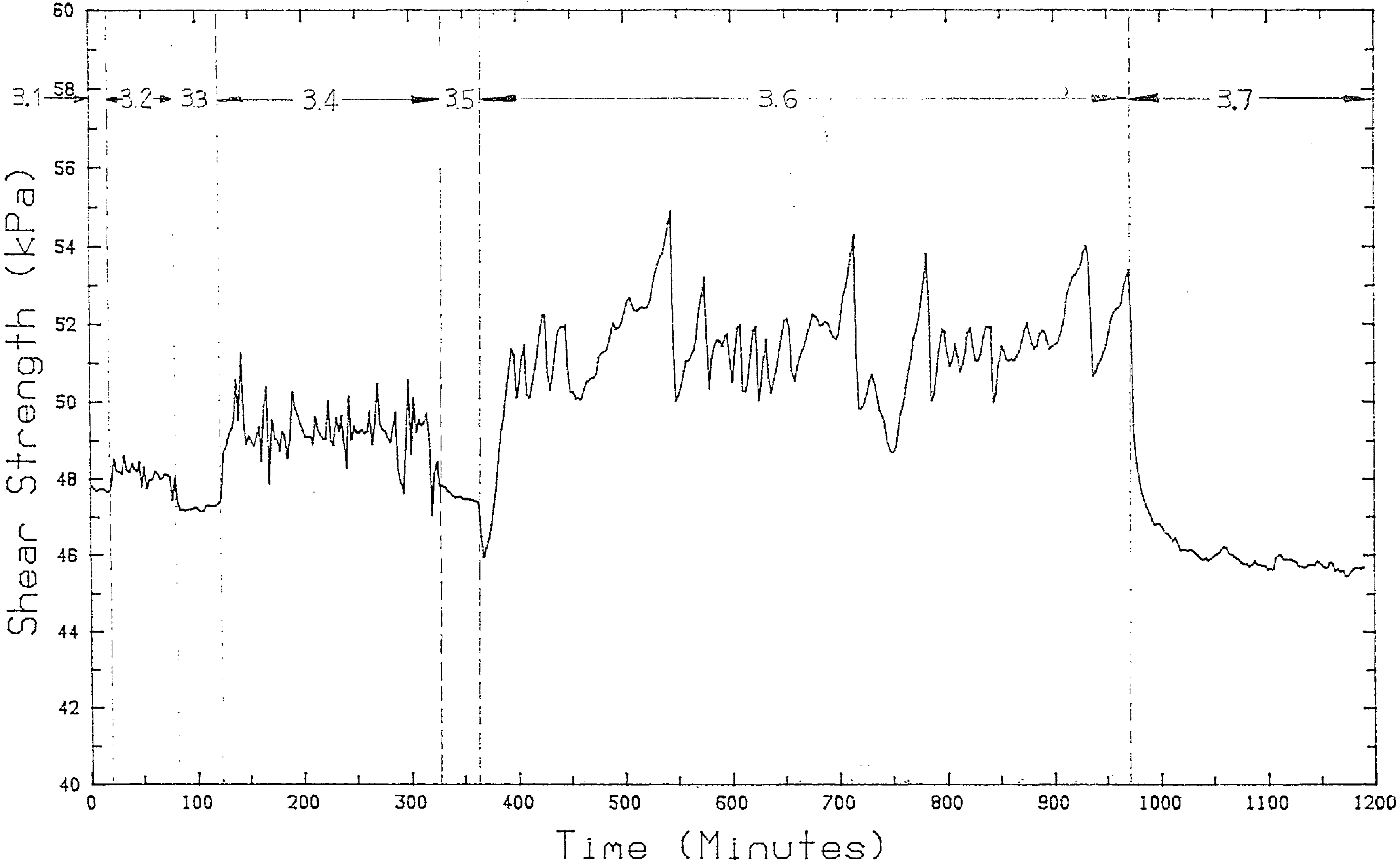
* An additional run of 150mm at Rate #4 was initiated in an attempt to get some degradation rate data at slower rates.

+ These statistics have been determined only over the range in which the strength had stabilised.

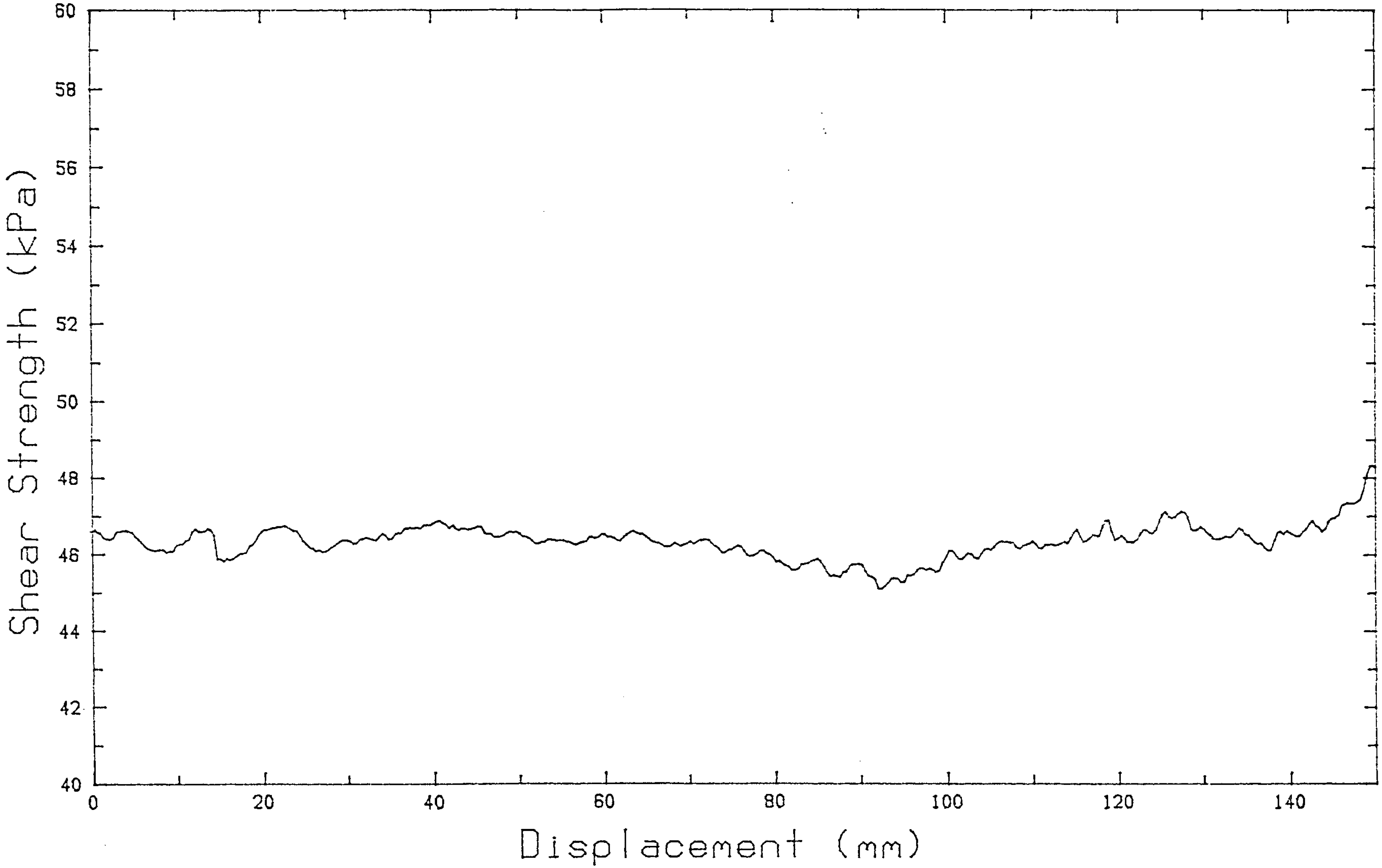

Ring Shear Test # 2 Run 1 0-10m Speed 7
Shear Strength (kPa)
Displacement (m)
40
42
44
46
48
50
52
54
56
58
60
0
1
2
3
4
5
6
7
8
9
10


Ring Shear Test # 2 Run 2 Medium Rate Study
2.1
2.2
2.3
2.4
2.5
2.6
Shear Strength (kPa)
60
58
56
54
52
50
48
46
44
42
40
0
20
40
60
80
100
120
140
160
180
200
220
240
Time (Minutes)

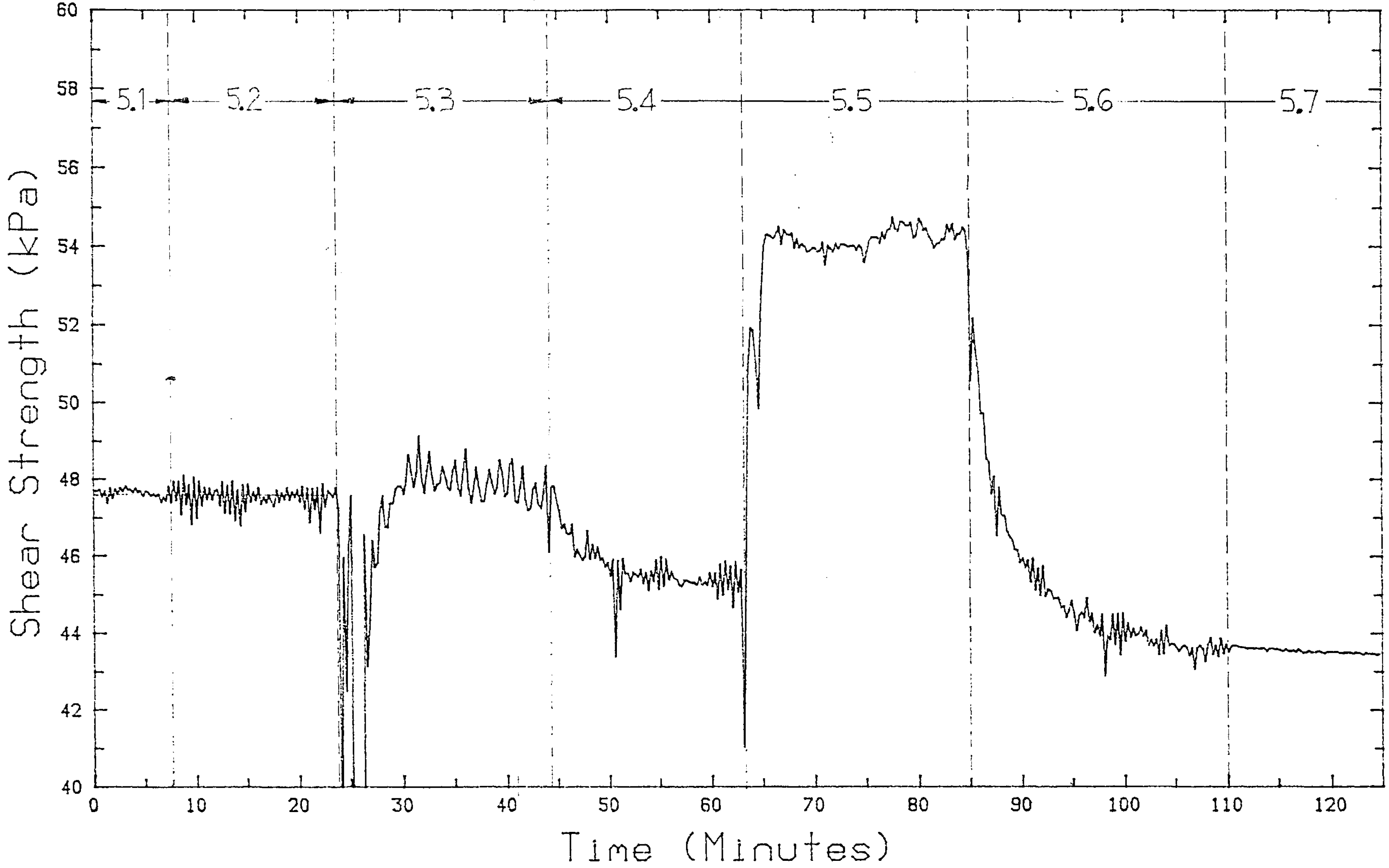
Ring Shear Test # 2 Run 3 Slow Rate Study



Ring Shear Test # 2 Run 4 Speed 4



Ring Shear Test # 2 Run 5 Fast Rate Study



## RING SHEAR TEST #3

Soil : Temuka Clay

Batch : 5

Date of Test : 2nd October 1989

Normal Stress : 200 kPa

Overconsolidation Ratio : 3

Initial Moisture Content : 28.2 %

Initial Sample Depth : 6.603 mm

### Consolidation Stages

| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 16 kPa | 6.443 mm |
| 2 | 42 kPa | 6.178 mm |
| 3 | 94 kPa | 5.839 mm |
| 4 | 200 kPa | 5.437 mm |
| 5 | 408 kPa | 4.958 mm |
| 6 | 600 kPa | 4.615 mm |
| 7 | 200 kPa | 4.771 mm |

### Shearing Stages

| STAGE | SHEARING RATES | SAMPLE DEPTH |
|---|---|---|
| 1 | #7 | 2.186 mm |
| 2 | #4, #5, #6 | 1.816 mm |
| 3 | #1, #2, #3, #4 | 1.809 mm |
| 4 | #8, #9 | 0.748 mm |

Final Moisture Content : 32.2 %

Table A4.3a TEST #3 PROGRAMME

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 0:00 | Rate 7 ON | 0.000 | | | |
| 2:29 | Rate 4t FOR | | | | |
| 2:32 | Rate 4t REV | | 1 | #7 46 | 220 |
| 2:54 | Rate 4t FOR | | | | |
| 3:37 | Rate 4t REV | | | | |
| 3:40 | | 10.120 | | | |
| 4:23 | Rate 4t FOR | | 2.1 | #7 46 | 45 |
| 4:25 | Rate 7 OFF | 12.190 | | | |
| | | | 2.2 | #4 .124 | 36 |
| 5:01 | Rate 6 ON | 12.194 | | | |
| 5:02 | Rate 4t REV | | | | |
| | | | 2.3 | #6 7.0 | 33 |
| 5:34 | Rate 4t FOR | | | | |
| 5:40 | Rate 6 OFF | 12.425 | | | |
| | | | 2.4 | #4 .124 | 42 |
| 6:22 | Rate 5 ON | 12.431 | | | |
| 6:23 | Rate 4t REV | | | | |
| | | | 2.5 | #5 0.93 | 37 |
| 6:59 | Rate 4t FOR | | | | |
| 7:00 | Rate 5 OFF | 12.465 | | | |
| | | | 2.6 | #4 .124 | 38 |
| 7:38 | Rate 4p ON | 12.470 | | | |
| 7:39 | Rate 4t OFF | | | | |
| 7:42 | Rate 3 REV | | 3.1 | #4 .124 | 25 |
| 8:12 | Rate 3 FOR | | | | |
| 8:13 | Rate 4p OFF | 12.473 | | | |
| | | | 3.2 | #3 .018 | 47 |
| 9:00 | Rate 4p ON | 12.474 | | | |
| 9:01 | Rate 3 REV | | | | |
| | | | 3.3 | #4 .124 | 62 |
| 9:59 | Rate 3 OFF | | | | |
| 10:00 | Rate 2 FOR | | | | |
| 10:02 | Rate 4p OFF | 12.481 | | | |
| | | | 3.4 | #2 .0022 | 205 |
| 13:26 | Rate 4p ON | 12.482 | | | |
| 13:27 | Rate 2 OFF | | | | |
| | | | 3.5 | #4 .124 | 40 |
| 13:28 | Rate 1 FOR | | | | |
| 14:06 | Rate 4p OFF | 12.487 | | | |
| | | | 3.6 | #1 .00029 | 584 |
| 23:50 | Rate 4p ON | 12.487 | | | |
| 23:51 | Rate 1 OFF | | | | |
| 23:54 | Rate 3 REV | | 3.7 | #4 .124 | 47 |

Table A4.3a TEST #3 PROGRAMME (continued)

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 24:39 | Rate 3 OFF | | | | |
| 24:40 | Rate 4t FOR | | | | |
| 24:41 | Rate 4p OFF | 12.493 | | | |
| | | | 4.1 | #4 .124 | 20 |
| 25:01 | Rate 7 ON | 12.495 | | | |
| 25.02 | Rate 4t REV | | | | |
| | | | 4.2 | #7 46 | 26 |
| 25:26 | Rate 4t FOR | | | | |
| 25:27 | Rate 7 OFF | 13.691 | | | |
| | | | 4.3 | #4 .124 | 19 |
| 25:46 | Rate 8 ON | 13.694 | | | |
| 25:47 | Rate 4t REV | | | | |
| | | | 4.4 | #8 310 | 24 |
| 26:09 | Rate 4t FOR | | | | |
| 26:10 | Rate 8 OFF | 21.134 | | | |
| | | | 4.5 | #4 .124 | 22 |
| 26:32 | Rate 9 ON | 21.136 | | | |
| 26:33 | Rate 4t REV | | | | |
| | | | 4.6 | #9 2079 | 15 |
| 26:46 | Rate 4t FOR | | | | |
| 26:47 | Rate 9 OFF | 52.321 | | | |
| | | | 4.7 | #4 .124 | 29 |
| 27:16 | Rate 4t OFF | 52.325 | | | |

Table A4.3b TEST #3 SAMPLE DEGRADATION AND WATER DEMAND

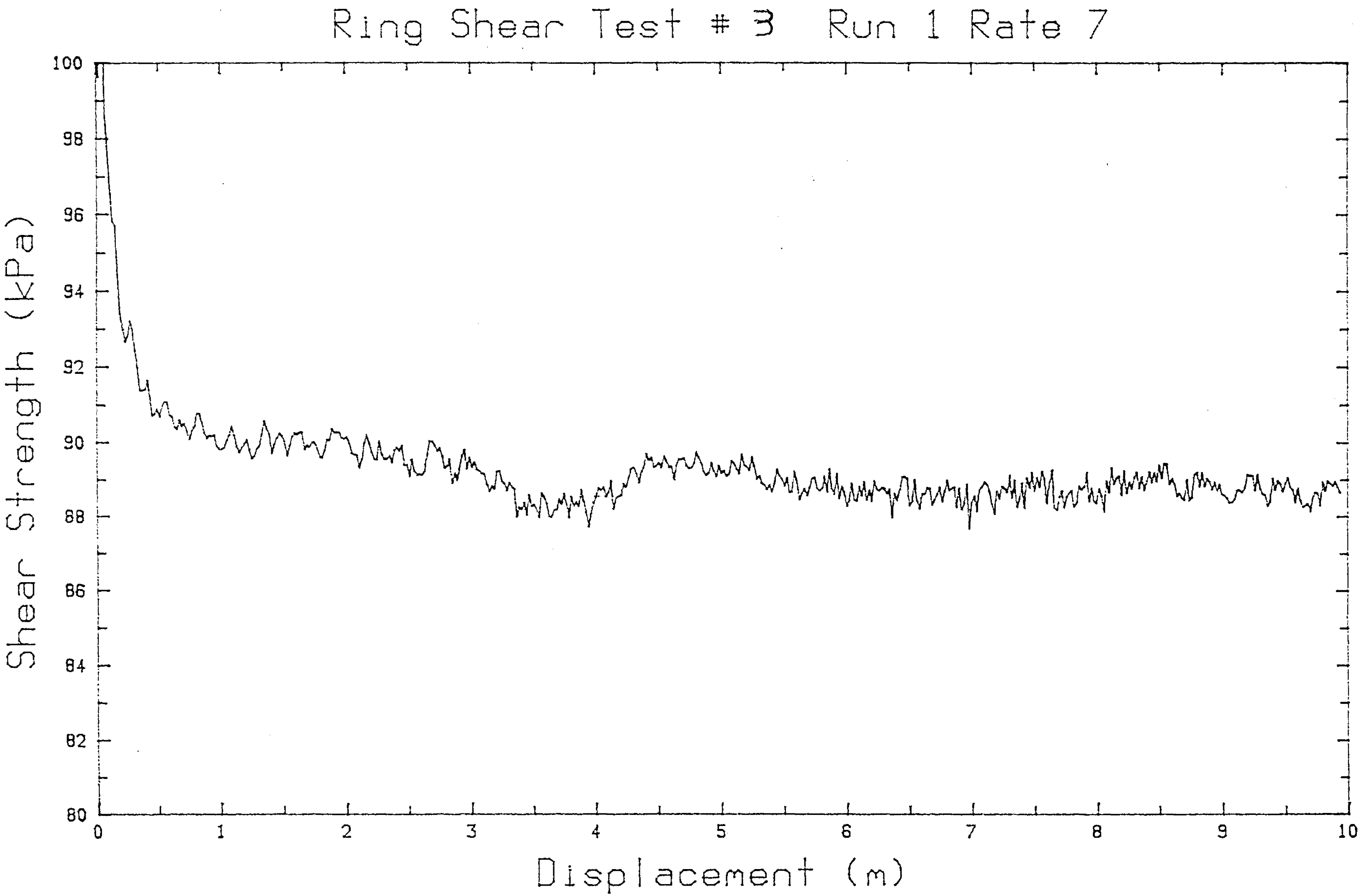
| Run No. | Shear Rate (mm/min) | Duration (min) | Δ Depth (mm) | Water Demand (ml) |
|---|---|---|---|---|
| 1.0 | #7 46.0 | 220 | 2.585 | 0.99 |
| 2.1 | #7 46.0 | 45 | 0.256 | 0.05 |
| 2.2 | #4t 0.124 | 36 | 0.000 | 0.00 |
| 2.3 | #6 7.0 | 33 | 0.041 | 0.00 |
| 2.4 | #4t 0.124 | 42 | 0.015 | 0.00 |
| 2.5 | #5 0.93 | 37 | 0.008 | 0.00 |
| 2.6 | #4t 0.124 | 38 | 0.004 | 0.00 |
| 3.1 | #4p 0.124 | 25 | 0.002 | 0.00 |
| 3.2 | #3 0.018 | 47 | 0.001 | 0.00 |
| 3.3 | #4p 0.124 | 62 | 0.001 | 0.00 |
| 3.4 | #2 0.0022 | 205 | 0.000 | 0.00 |
| 3.5 | #4p 0.124 | 40 | 0.000 | 0.00 |
| 3.6 | #1 0.00029 | 584 | 0.002 | 0.00 |
| 3.7 | #4p 0.124 | 47 | 0.002 | 0.00 |
| 4.1 | #4t 0.124 | 20 | 0.000 | 0.00 |
| 4.2 | #7 46.0 | 26 | 0.107 | 0.00 |
| 4.3 | #4t 0.124 | 19 | 0.008 | 0.00 |
| 4.4 | #8 310 | 24 | 0.254 | 0.16 |
| 4.5 | #4t 0.124 | 22 | 0.002 | 0.01 |
| 4.6 | #9 2079 | 15 | 0.993 | 0.30 |
| 4.7 | #4t 0.124 | 29 | 0.073 | 0.03 |

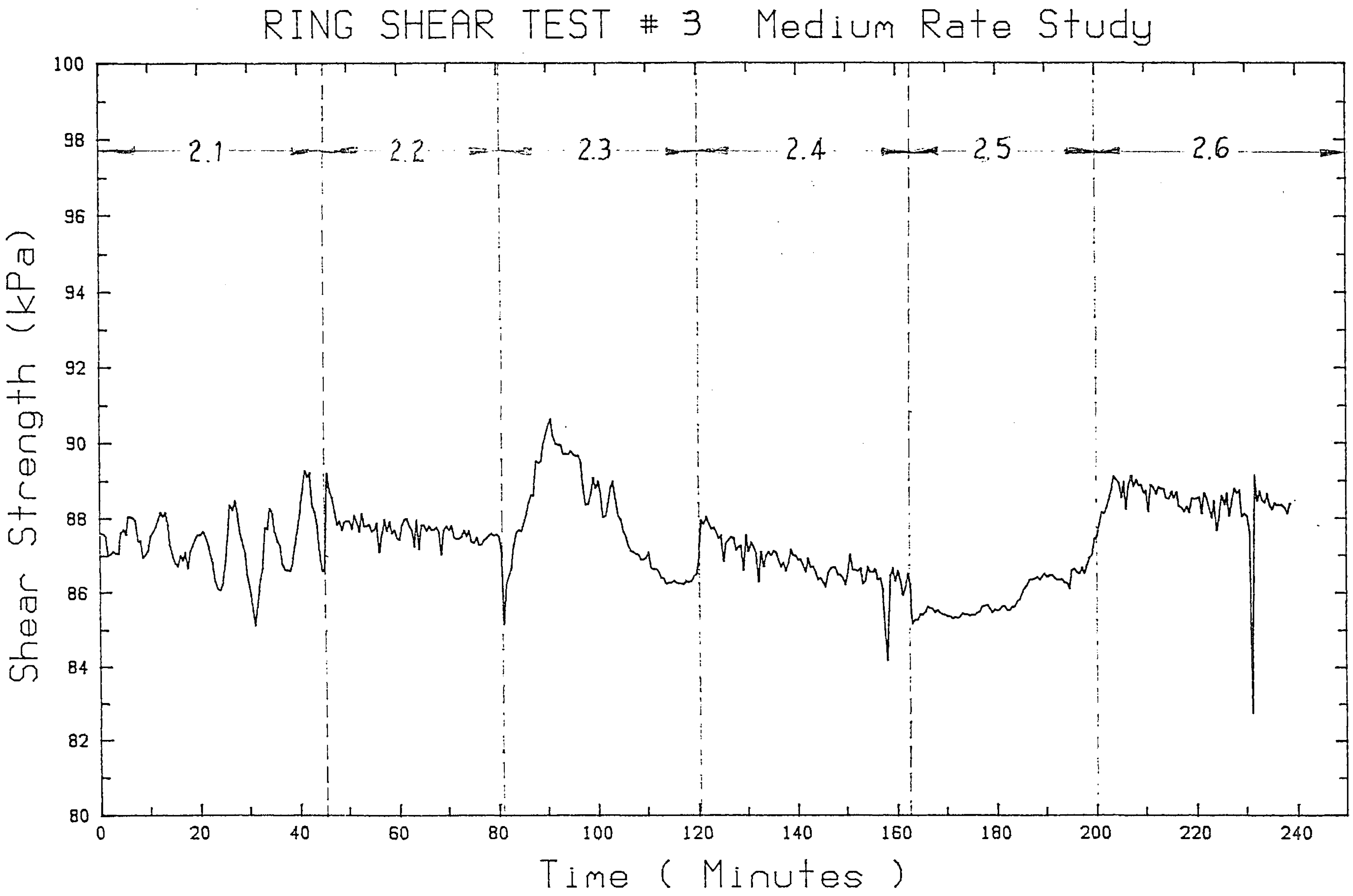
Table A4.3c ANALYSIS OF TEST #3 RESULTS

| Run No. | Shear Rate (mm/min) | | Duration (min) | Average $\tau$ (kPa) | Stand. Dev. $\tau$ (kPa) |
|---|---|---|---|---|---|
| 1.0 | #7 | 46.0 | 220 | 88.763 | 0.499 |
| 2.1 | #7 | 46.0 | 45 | 87.354 | 0.867 |
| 2.2 | #4t | 0.124 | 36 | 87.691 | 1.289 |
| 2.3 | #6 | 7.0 | 33 | 87.949 | 1.406 |
| 2.4 | #4t | 0.124 | 42 | 86.788 | 1.398 |
| 2.5 | #5 | 0.93 | 37 | 86.189 | 1.127 |
| 2.6 | #4t | 0.124 | 38 | 88.413 | 1.483 |
| 3.1 | #4p | 0.124 | 25 | 88.312 | 0.246 |
| 3.2 | #3 | 0.018 | 47 | 90.383 | 1.784 |
| 3.3 | #4p | 0.124 | 62 | 88.838 | 0.434 |
| 3.4 | #2 | 0.0022 | 205 | 93.228 + | 1.459 + |
| 3.5 | #4p | 0.124 | 40 | 90.853 | 0.316 |
| 3.6 | #1 | 0.00029 | 584 | 97.471 + | 1.666 + |
| 3.7 | #4p | 0.124 | 47 | 91.962 | 1.061 |
| 4.1 | #4t | 0.124 | 20 | 93.185 | 1.200 |
| 4.2 | #7 | 46.0 | 26 | 88.751 | 1.340 |
| 4.3 | #4t | 0.124 | 19 | 89.192 | 1.342 |
| 4.4 | #8 | 310 | 24 | 90.920 | 2.524 |
| 4.5 | #4t | 0.124 | 22 | 87.953 + | 1.315 + |
| 4.6 | #9 | 2079 | 15 | 97.636 | 2.698 |
| 4.7 | #4t | 0.124 | 29 | * | * |

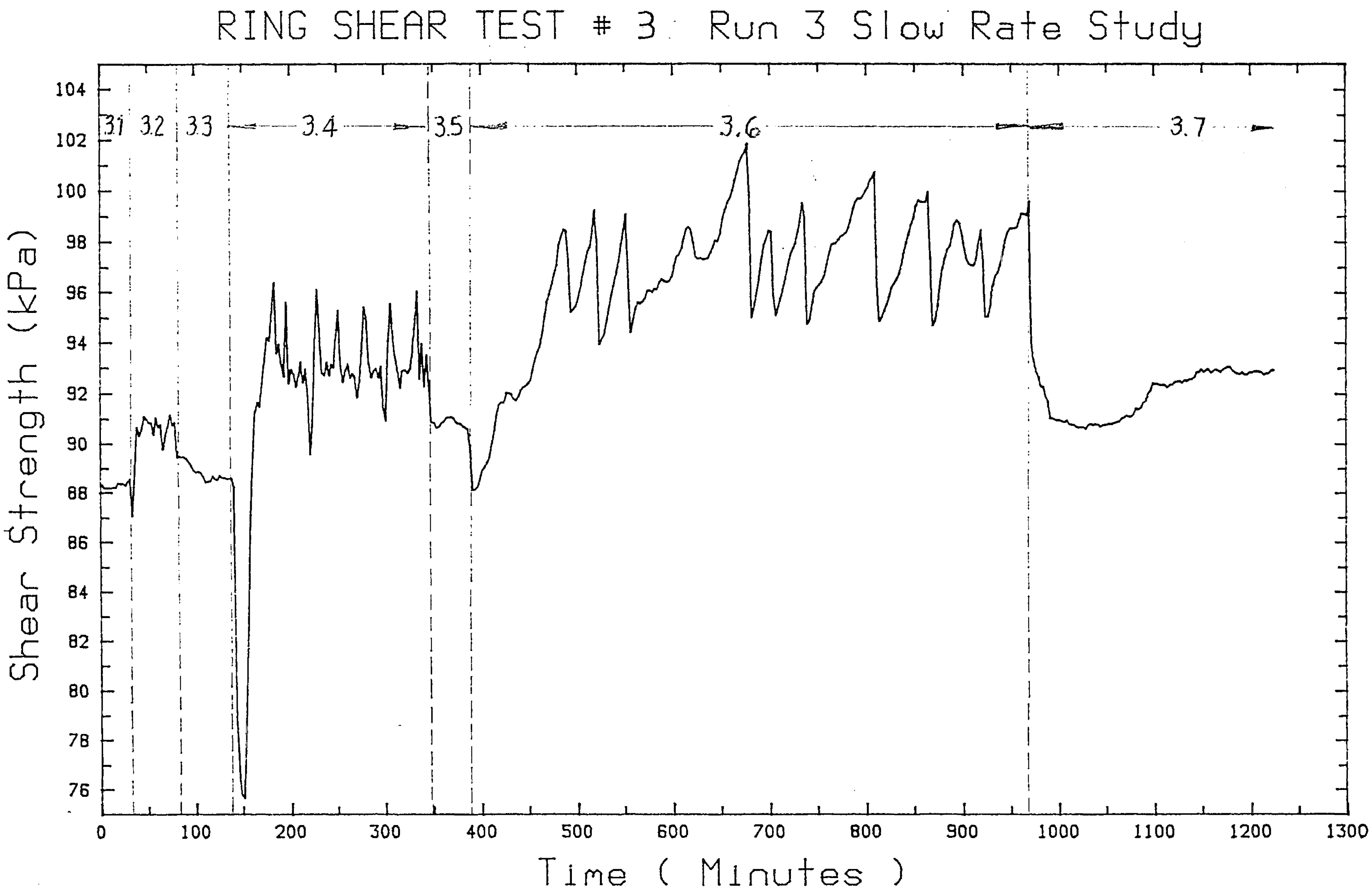
* After 29 minutes the strength was still trending done, however, it was not possible to continue shearing as the depth of soil remaining was insufficient.

+ These statistics have been determined only over the range in which the strength had stabilised.


Ring Shear Test # 3 Run 1 Rate 7
Shear Strength (kPa)
100
98
96
94
92
90
88
86
84
82
80
0
1
2
3
4
5
6
7
8
9
10
Displacement (m)

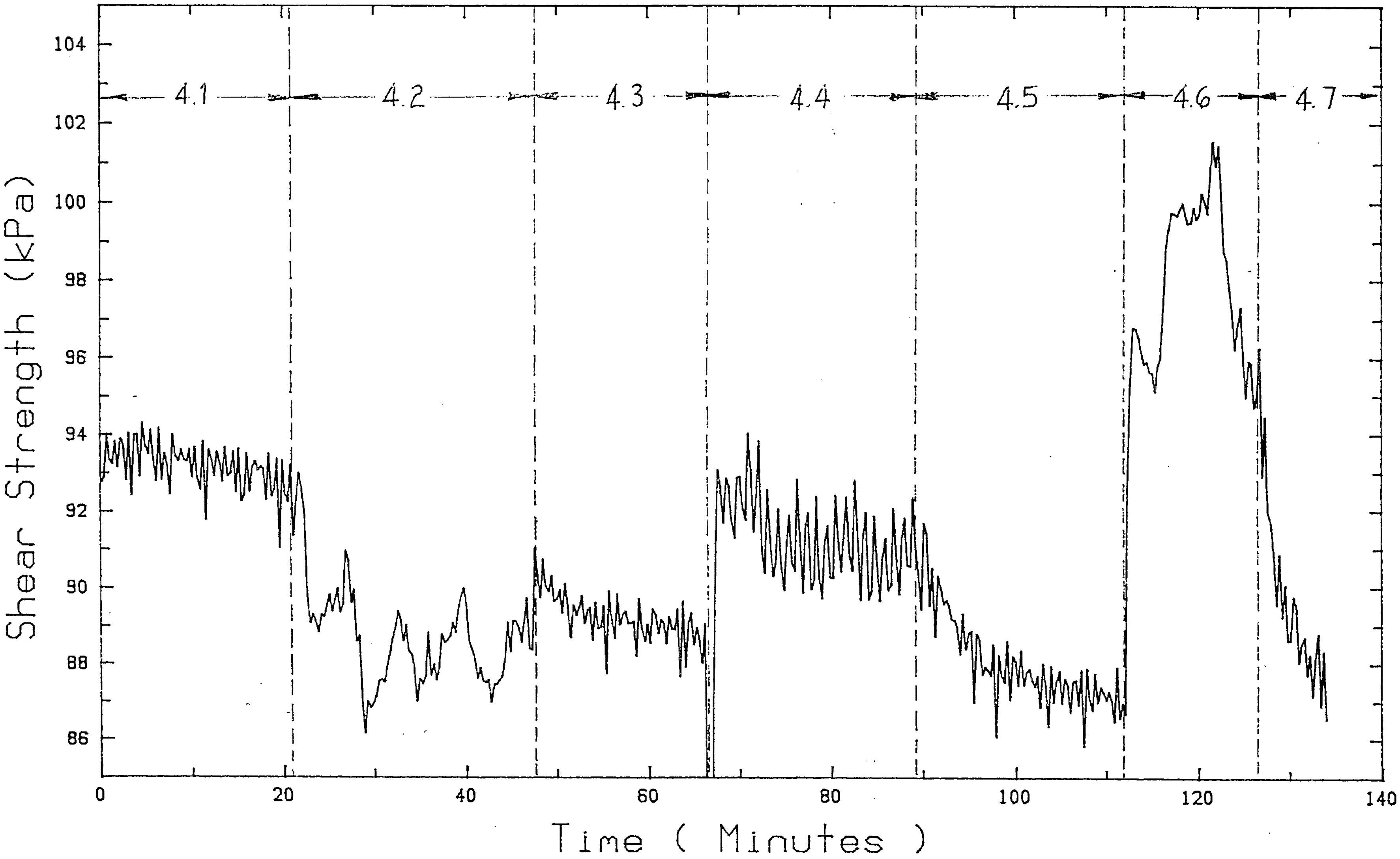


RING SHEAR TEST # 3 Medium Rate Study
Shear Strength (kPa)
Time ( Minutes )
2.1
2.2
2.3
2.4
2.5
2.6
100
98
96
94
92
90
88
86
84
82
80
0
20
40
60
80
100
120
140
160
180
200
220
240



RING SHEAR TEST # 3 Run 3 Slow Rate Study
Shear Strength (kPa)
Time ( Minutes )
3.1
3.2
3.3
3.4
3.5
3.6
3.7
104
102
100
98
96
94
92
90
88
86
84
82
80
78
76
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300

RING SHEAR TEST # 3 Fast Rate Study



RING SHEAR TEST #4

Soil : Temuka Clay

Batch : 5

Date of Test : 4th October 1989

Normal Stress : 400 kPa

Overconsolidation Ratio : 3

Initial Moisture Content : 27.7 %

Initial Sample Depth : 6.841 mm

Consolidation Stages

| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 16 kPa | 6.535 mm |
| 2 | 42 kPa | 6.277 mm |
| 3 | 94 kPa | 5.977 mm |
| 4 | 200 kPa | 5.612 mm |
| 5 | 408 kPa | 5.179 mm |
| 6 | 826 kPa | 4.661 mm |
| 7 | 1200 kPa | 4.273 mm |
| 8 | 400 kPa | 4.463 mm |

Shearing Stages

| STAGE | SHEARING RATES | SAMPLE DEPTH |
|---|---|---|
| 1 | #7 | 1.801 mm |
| 2 | #4, #5, #6 | 1.574 mm |
| 3 | #1, #2, #3, #4 | 1.562 mm |
| 4 | #8, #9 | 0.000 mm |

Final Moisture Content : 30.6 %

Table A4.4a TEST #4 PROGRAMME

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 0:00 | Rate 7 ON | 0.000 | ------- | ----------- | -------- |
| 2:10 | Rate 4t REV | | | | |
| 2:32 | Rate 4t FOR | | 1 | #7 46 | 217 |
| 3:15 | Rate 4t REV | | | | |
| 3:37 | | 10.000 | ------- | ----------- | -------- |
| 3:56 | Rate 4t FOR | | 2.1 | #7 46 | 20 |
| 3:57 | Rate 7 OFF | 10.920 | ------- | ----------- | -------- |
| | | | 2.2 | #4 .124 | 64 |
| 5:01 | Rate 6 ON | 10.928 | ------- | ----------- | -------- |
| 5:02 | Rate 4t REV | | | | |
| | | | 2.3 | #6 7.0 | 41 |
| 5:41 | Rate 4t FOR | | | | |
| 5:42 | Rate 6 OFF | 11.215 | ------- | ----------- | -------- |
| | | | 2.4 | #4 .124 | 36 |
| 6:18 | Rate 5 ON | 11.219 | ------- | ----------- | -------- |
| 6:19 | Rate 4t REV | | | | |
| | | | 2.5 | #5 0.93 | 39 |
| 6:56 | Rate 4t FOR | | | | |
| 6:57 | Rate 5 OFF | 11.256 | ------- | ----------- | -------- |
| 7:16 | Rate 4p ON | | 2.6 | #4 .124 | 42 |
| 7:17 | Rate 4t OFF | | | | |
| 7:39 | | 11.261 | ------- | ----------- | -------- |
| 8:01 | Rate 2 ON | | 3.1 | #4 .124 | 25 |
| 8:04 | Rate 4p OFF | 11.264 | ------- | ----------- | -------- |
| | | | 3.2 | #2 .0022 | 330 |
| 13:34 | Rate 4p ON | 11.265 | ------- | ----------- | -------- |
| 13:35 | Rate 1 ON | | 3.3 | #4 .124 | 30 |
| 14:04 | Rate 4p OFF | 11.268 | ------- | ----------- | -------- |
| | | | 3.4 | #1 .00029 | 715 |
| 25:59 | Rate 4p ON | 11.269 | ------- | ----------- | -------- |
| 26:00 | Rate 1 OFF | | | | |
| 26:08 | Rate 3t REV | | 3.5 | #4 .124 | 175 |
| 28:52 | Rate 3t FOR | | | | |
| 28:54 | Rate 4p OFF | 11.290 | ------- | ----------- | -------- |
| | | | 3.6 | #3 .0022 | 80 |
| 30:14 | Rate 4p ON | 11.291 | ------- | ----------- | -------- |

Table A4.4a TEST #4 PROGRAMME (continued)

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 30:16 | Rate 3t REV | | | | |
| 30:46 | Rate 3t OFF | | 4.1 | #4 .124 | 42 |
| 30:48 | Rate 4t ON | | | | |
| 30:49 | Rate 4p OFF | | | | |
| 30:56 | Rate 7 ON | 11.296 | | | |
| 30:57 | Rate 4t REV | | | | |
| | | | 4.2 | #7 46 | 21 |
| 31:16 | Rate 4t FOR | | | | |
| 31:17 | Rate 7 OFF | 12.262 | | | |
| | | | 4.3 | #4 .124 | 22 |
| 31:39 | Rate 8 ON | 12.264 | | | |
| 31:40 | Rate 4t REV | | | | |
| | | | 4.4 | #8 310 | 19 |
| 31:57 | Rate 4t FOR | | | | |
| 31:58 | Rate 8 OFF | 18.154 | | | |
| | | | 4.5 | #4 .124 | 20 |
| 32:18 | Rate 9 ON | 18.157 | | | |
| 32:19 | Rate 4t REV | | | | |
| | | | 4.6 | #9 2079 | 16 |
| 32:34 | Rate 9 OFF | 51.421 | | | |
| | | | 4.7 | #4 .124 | 0 |
| 32:34 | Rate 4t OFF | 51.421 | | | |

Table A4.4b TEST #4 SAMPLE DEGRADATION AND WATER DEMAND

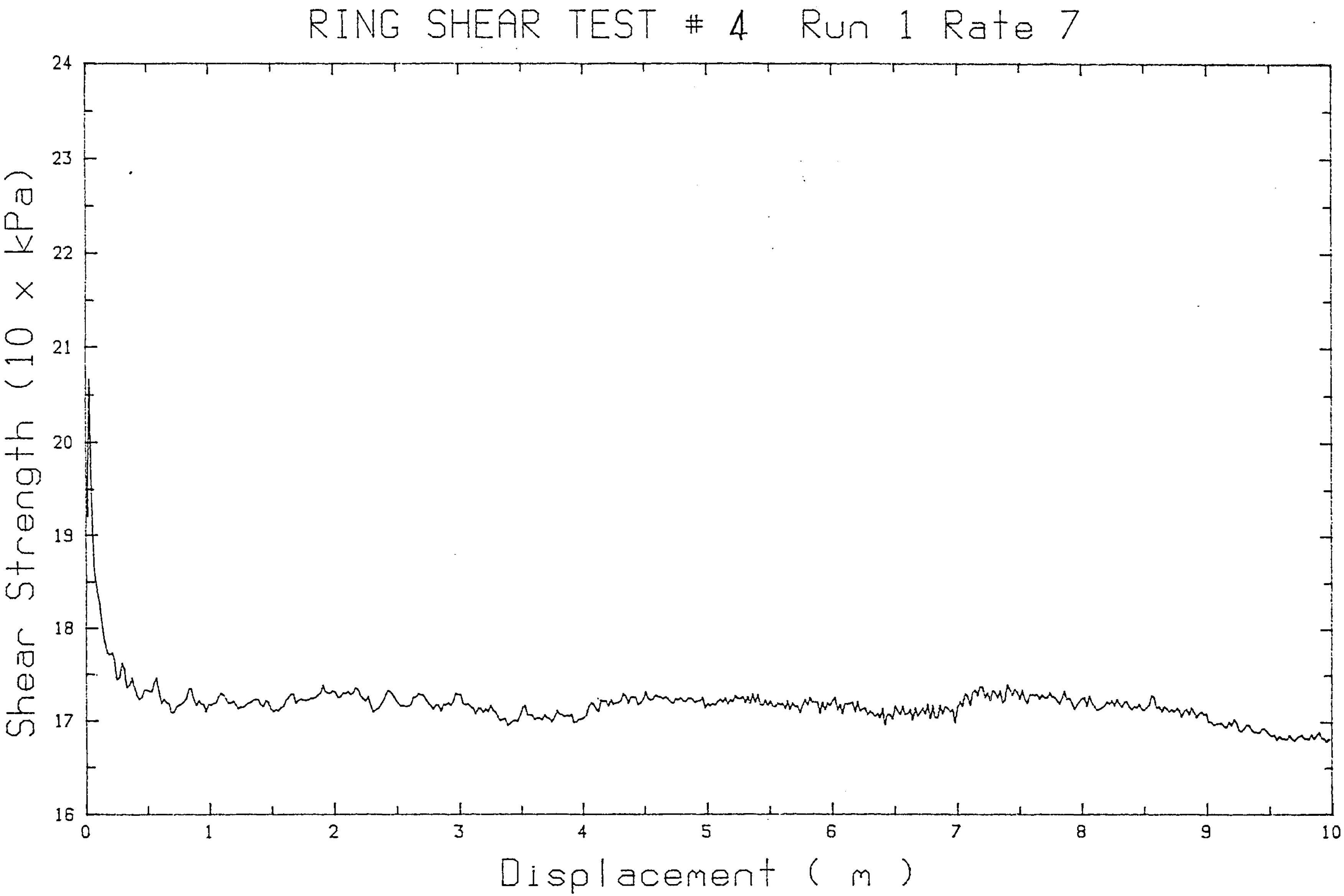
| Run No. | Shear Rate (mm/min) | Duration (min) | Δ Depth (mm) | Water Demand (ml) |
|---|---|---|---|---|
| 1.0 | #7 46.0 | 217 | 2.670 | 1.18 |
| 2.1 | #7 46.0 | 20 | 0.098 | 0.06 |
| 2.2 | #4t 0.124 | 64 | 0.041 | 0.03 |
| 2.3 | #6 7.0 | 41 | 0.038 | 0.02 |
| 2.4 | #4t 0.124 | 36 | 0.019 | 0.01 |
| 2.5 | #5 0.93 | 39 | 0.006 | 0.00 |
| 2.6 | #4tp 0.124 | 42 | 0.000 | 0.01 |
| 3.1 | #4p 0.124 | 25 | 0.006 | 0.00 |
| 3.2 | #2 0.0022 | 330 | 0.000 | -0.02 |
| 3.3 | #4p 0.124 | 30 | 0.000 | 0.00 |
| 3.4 | #1 0.00029 | 715 | 0.006 | -0.02 |
| 3.5 | #4p 0.124 | 175 | 0.000 | 0.00 |
| 3.6 | #3 0.018 | 80 | 0.000 | 0.00 |
| 4.1 | #4t 0.124 | 42 | 0.011 | -0.02 |
| 4.2 | #7 46.0 | 21 | 0.135 | 0.02 |
| 4.3 | #4t 0.124 | 22 | 0.053 | 0.00 |
| 4.4 | #8 310 | 19 | 0.265 | 0.13 |
| 4.5 | #4t 0.124 | 20 | 0.160 | 0.04 |
| 4.6 | #9 2079 | 16 | 1.023 | 0.18 |

Table A4.4c  ANALYSIS OF TEST #4 RESULTS

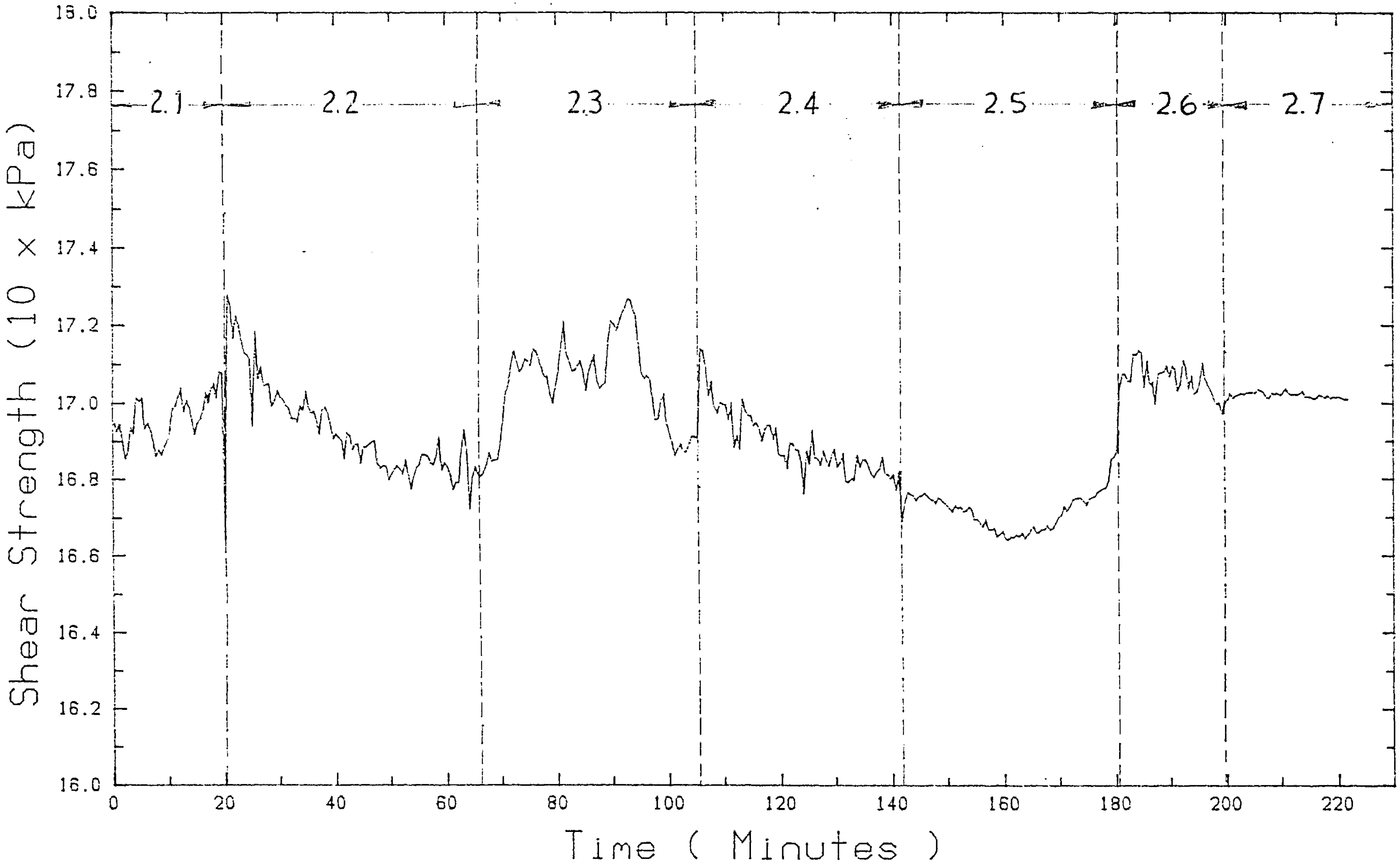
| Run No. | Shear Rate (mm/min) | Duration (min) | Average $\tau$ (kPa) | Stand. Dev. $\tau$ (kPa) |
|---|---|---|---|---|
| 1.0 | #7 46.0 | 217 | 171.48 | 1.149 |
| 2.1 | #7 46.0 | 20 | 169.61 | 0.911 |
| 2.2 | #4t 0.124 | 64 | 169.33 | 2.392 |
| 2.3 | #6 7.0 | 41 | 170.34 | 1.312 |
| 2.4 | #4t 0.124 | 36 | 168.94 | 2.156 |
| 2.5 | #5 0.93 | 39 | 167.16 | 0.640 |
| 2.6 | #4tp 0.124 | 42 | 170.38 | 1.414 |
| 3.1 | #4p 0.124 | 25 | 170.11 | 0.245 |
| 3.2 | #2 0.0022 | 330 | 174.65 | 2.578 |
| 3.3 | #4p 0.124 | 30 | 171.60 | 0.130 |
| 3.4 | #1 0.00029 | 715 | 179.13 + | 2.900 + |
| 3.5 | #4p 0.124 | 175 | 173.79 | 0.883 |
| 3.6 | #3 0.018 | 80 | 177.26 + | 1.939 + |
| 4.1 | #4t 0.124 | 42 | 174.73 | 0.983 |
| 4.2 | #7 46.0 | 21 | 174.64 | 2.625 |
| 4.3 | #4t 0.124 | 22 | 172.21 | 2.749 |
| 4.4 | #8 310 | 19 | 178.97 | 1.608 |
| 4.5 | #4t 0.124 | 20 | 173.25 | 2.713 |
| 4.6 | #9 2079 | 16 | 187.93 | 2.968 |

* After 5 minutes shearing at the fast rate the depth of soil had become too thin to continue shearing.

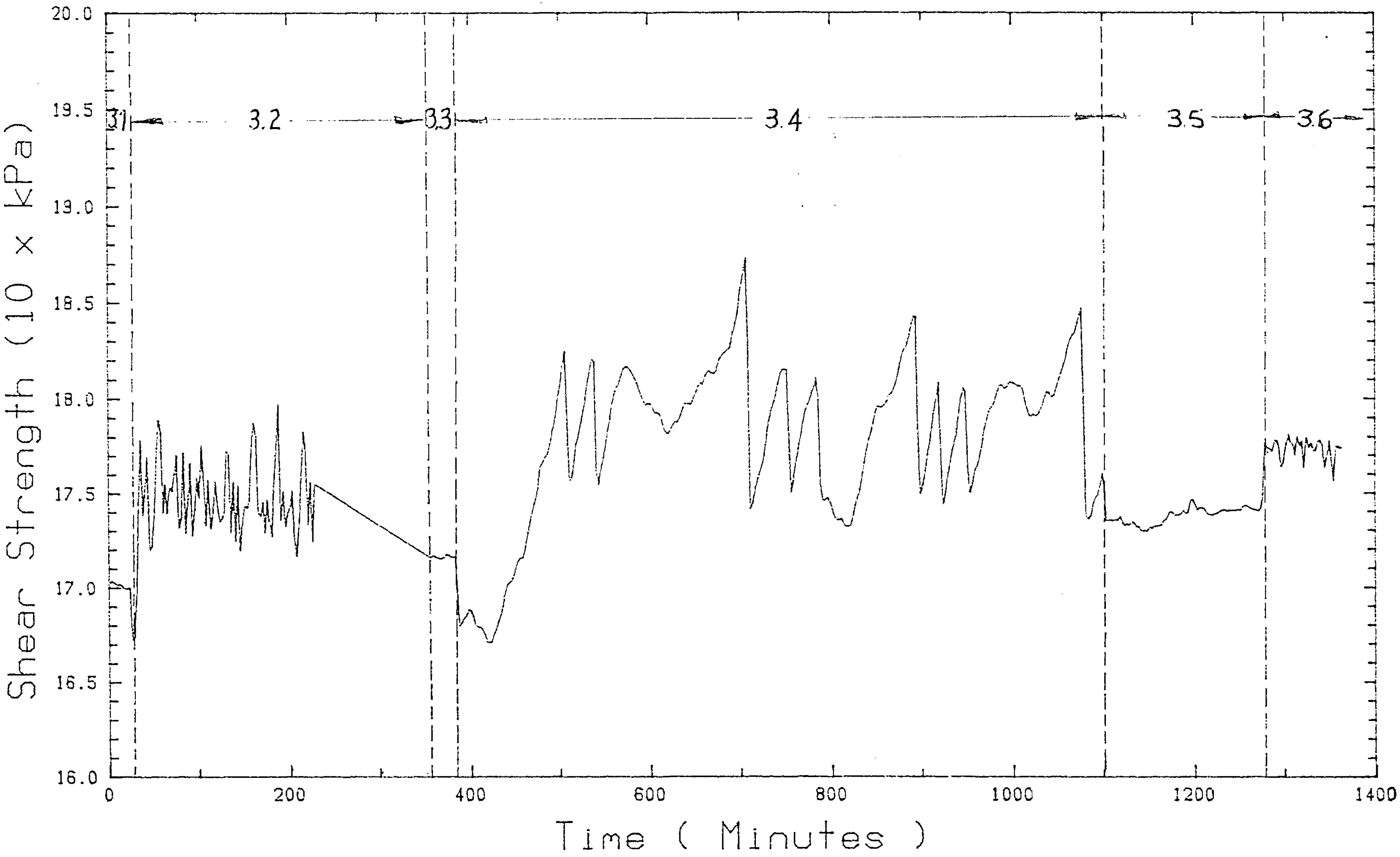
+ These averages have been taken only after the strength has stabilised.

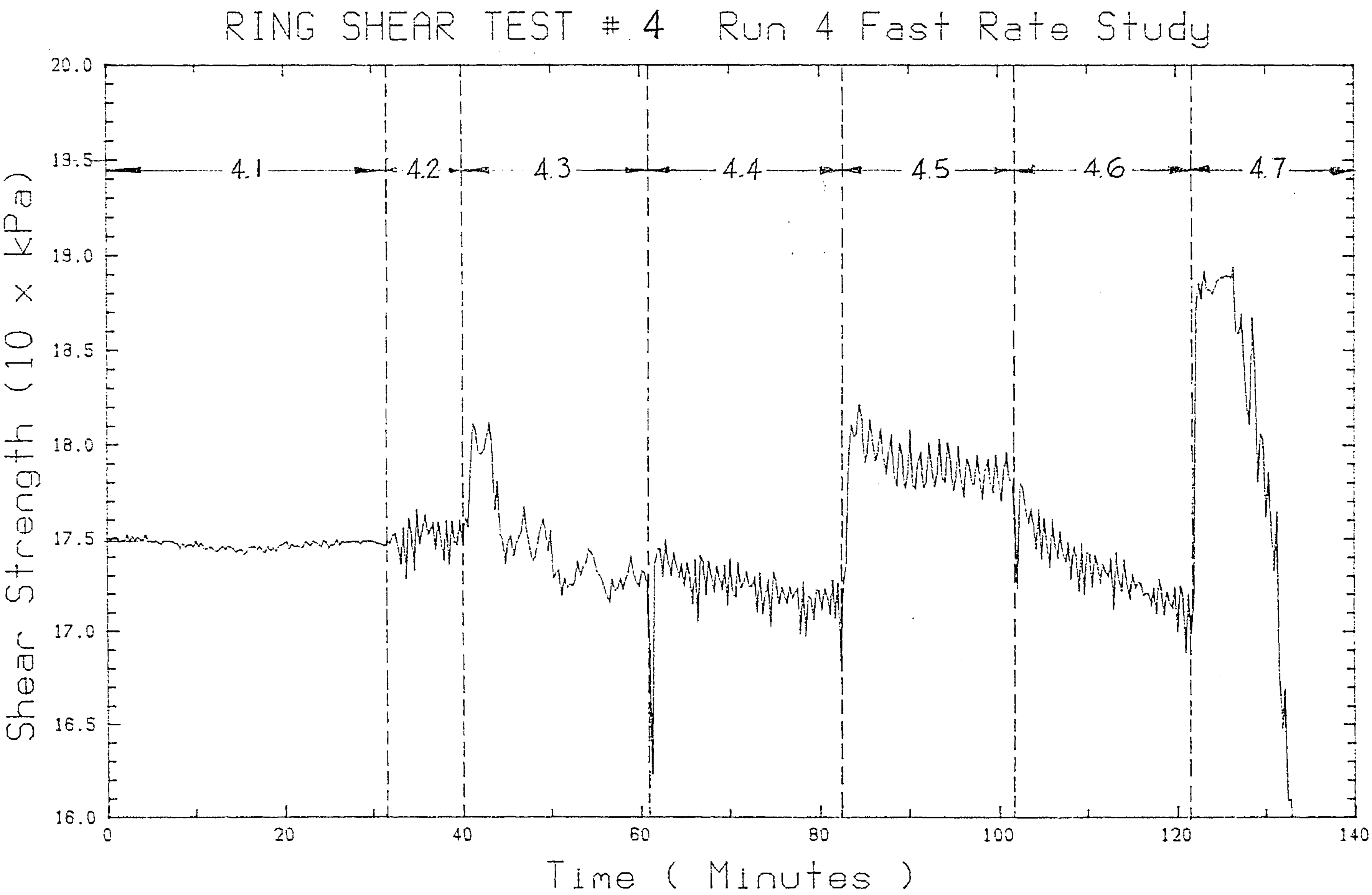

RING SHEAR TEST # 4 Run 1 Rate 7
Shear Strength (10 x kPa)
24
23
22
21
20
19
18
17
16
0
1
2
3
4
5
6
7
8
9
10
Displacement ( m )

RING SHEAR TEST # 4 Run 2 Medium Rate Study



RING SHEAR TEST # 4 Run 3 Slow Rate Study




RING SHEAR TEST # 4 Run 4 Fast Rate Study
4.1
4.2
4.3
4.4
4.5
4.6
4.7
Shear Strength (10 x kPa)
20.0
19.5
19.0
18.5
18.0
17.5
17.0
16.5
16.0
0
20
40
60
80
100
120
140
Time ( Minutes )

## RING SHEAR TEST #5

Soil : Temuka Clay

Batch : 5

Date of Test : 9th October 1989

Normal Stress : 50 kPa

Overconsolidation Ratio : 3

Initial Moisture Content : 27.9 %

Initial Sample Depth : 7.039 mm

### Consolidation Stages

| STAGE | TOTAL STRESS | SAMPLE DEPTH |
|---|---|---|
| 1 | 16 kPa | 6.806 mm |
| 2 | 42 kPa | 6.531 mm |
| 3 | 94 kPa | 6.087 mm |
| 4 | 150 kPa | 5.879 mm |
| 5 | 50 kPa | 5.965 mm |

### Shearing Stages

| STAGE | SHEARING RATES | SAMPLE DEPTH |
|---|---|---|
| 1 | #7 | 4.444 mm |
| 2 | #4, #5, #6 | 4.365 mm |
| 3 | #1, #2, #3, #4 | 4.360 mm |
| 4 | #8, #9 | 1.621 mm |

Final Moisture Content : 26.3 %

Table A4.5a TEST #5 PROGRAMME

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 0:00 | Rate 7 ON | 0.000 | | | |
| | | | 1 | #7 46 | 218 |
| 3:37 | | 10.000 | | | |
| 3:56 | Rate 4t FOR* | | 2.1 | #7 46 | 20 |
| 3:57 | Rate 7 OFF | 10.920 | | | |
| 5:03 | Rate 4t FOR | | 2.2 | #4 .124 | 20 |
| 5:17 | Rate 6 ON | 10.922 | | | |
| 5:18 | Rate 4t REV | | | | |
| | | | 2.3 | #6 7.0 | 35 |
| 5:51 | Rate 4t FOR | | | | |
| 5:52 | Rate 6 OFF | 11.167 | | | |
| | | | 2.4 | #4 .124 | 37 |
| 6:29 | Rate 5 ON | 11.172 | | | |
| 6:30 | Rate 4t REV | | | | |
| | | | 2.5 | #5 0.93 | 38 |
| 7:06 | Rate 4t FOR | | | | |
| 7:07 | Rate 5 OFF | 11.207 | | | |
| | | | 2.6 | #4 .124 | 23 |
| 7:30 | Rate 4p ON | 11.210 | | | |
| 7:31 | Rate 4t OFF | | | | |
| 7:33 | Rate 3 REV | | 3.1 | #4 .124 | 140 |
| 9:49 | Rate 3 FOR | | | | |
| 9:50 | Rate 4p OFF | 11.228 | | | |
| | | | 3.2 | #3 .018 | 119 |
| 13:09 | Rate 4p ON | 11.231 | | | |
| 13:10 | Rate 3 REV | | | | |
| | | | 3.3 | #4 .124 | 49 |
| 13:25 | Rate 3 OFF | | | | |
| 13:26 | Rate 1 ON | | | | |
| 13:58 | Rate 4p OFF | 11.237 | | | |
| | | | 3.4 | #1 .00029 | 622 |
| 24:20 | Rate 4p ON | 11.237 | | | |
| 24:21 | Rate 1 OFF | | | | |
| 24:24 | Rate 3 REV | | 3.5 | #4 .124 | 118 |
| 26:12 | Rate 3 OFF | | | | |
| 26:13 | Rate 2 ON ~ | | | | |
| 26:18 | Rate 4p OFF | 11.252 | | | |
| 29:37 | Rate 3 REV | | 3.6 | #2 .0022 | 200 |
| 29:38 | Rate 4p ON | 11.252 | | | |
| | | | 3.7 | #4 .124 | 60 |
| 30:38 | | 11.256 | | | |

* Clutch left out leading to drop of shear stress
~ Failure of D/A Power Supply

Table A4.5a TEST #5 PROGRAMME (continued)

| TIME LINE | | | SHEARING STAGES | | |
|---|---|---|---|---|---|
| Time H:MM | Event | Disp. m | Run # | Shear Rate mm/min | Duration min |
| 30:57 | Rate 3 OFF | | 4.1 | #4 .124 | 21 |
| 30:59 | Rate 7 ON | 11.259 | | | |
| 31:00 | Rate 4t REV | | | | |
| | | | 4.2 | #7 46 | 19 |
| 31:17 | Rate 4t FOR | | | | |
| 31:18 | Rate 7 OFF | 12.133 | | | |
| | | | 4.3 | #4 .124 | 20 |
| 31:38 | Rate 8 ON | 12.135 | | | |
| 31:39 | Rate 4t REV | | | | |
| | | | 4.4 | #8 310 | 20 |
| 31:57 | Rate 4t FOR | | | | |
| 31:58 | Rate 8 OFF | 18.335 | | | |
| | | | 4.5 | #4 .124 | 20 |
| 32:18 | Rate 9 ON | 18.338 | | | |
| 32:19 | Rate 4t REV | | | | |
| | | | 4.6 | #9 2079 | 20 |
| 32:37 | Rate 4t FOR | | | | |
| 32:38 | Rate 9 OFF | 59.918 | | | |
| 32:41 | Rate 4p ON | | | | |
| 33:43 | Rate 4t OFF | | 4.7 | #4 .124 | 28 |
| 33:06 | Rate 4p OFF | 59.921 | | | |

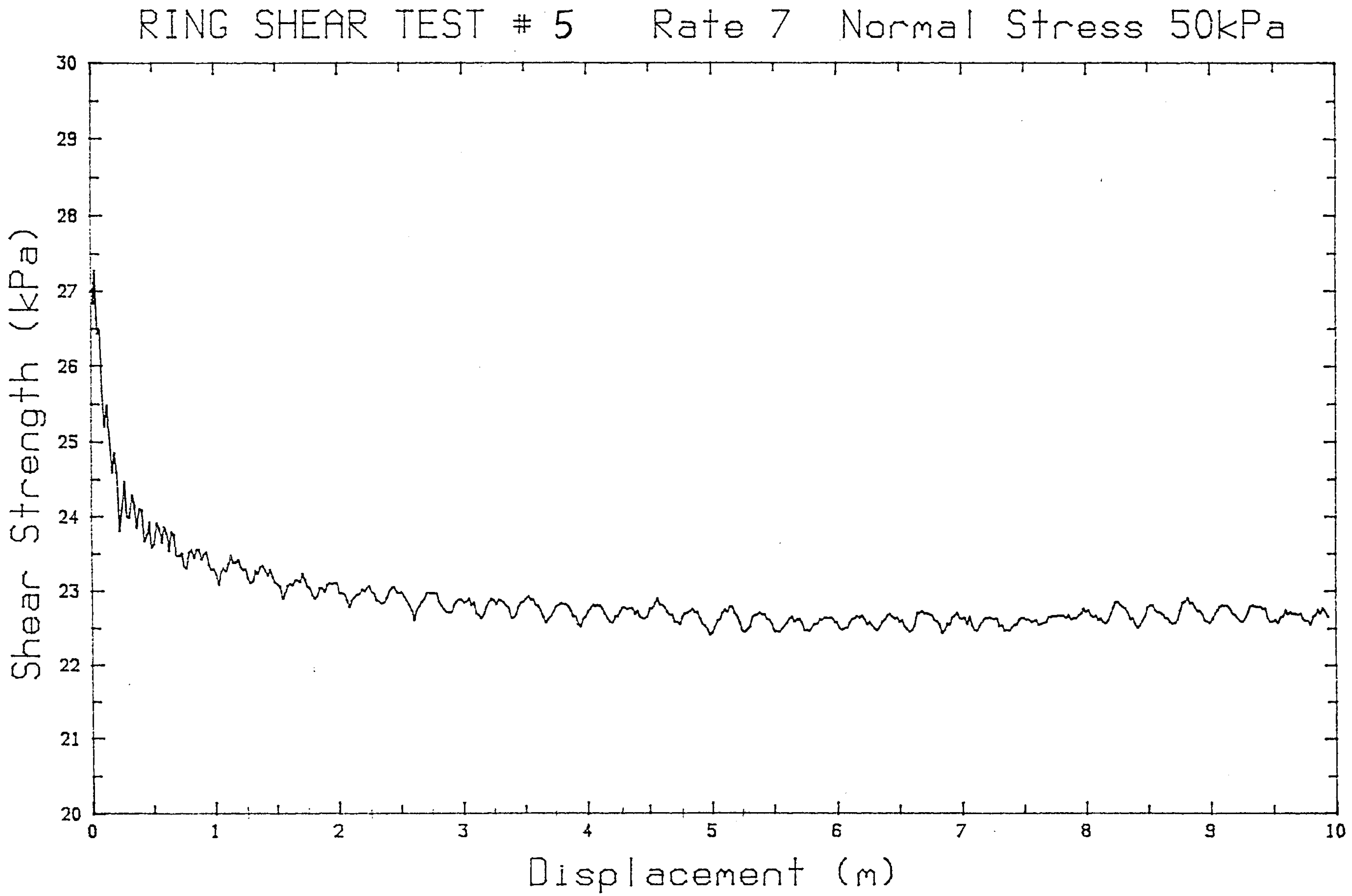
Table A4.5b TEST #5 SAMPLE DEGRADATION AND WATER DEMAND

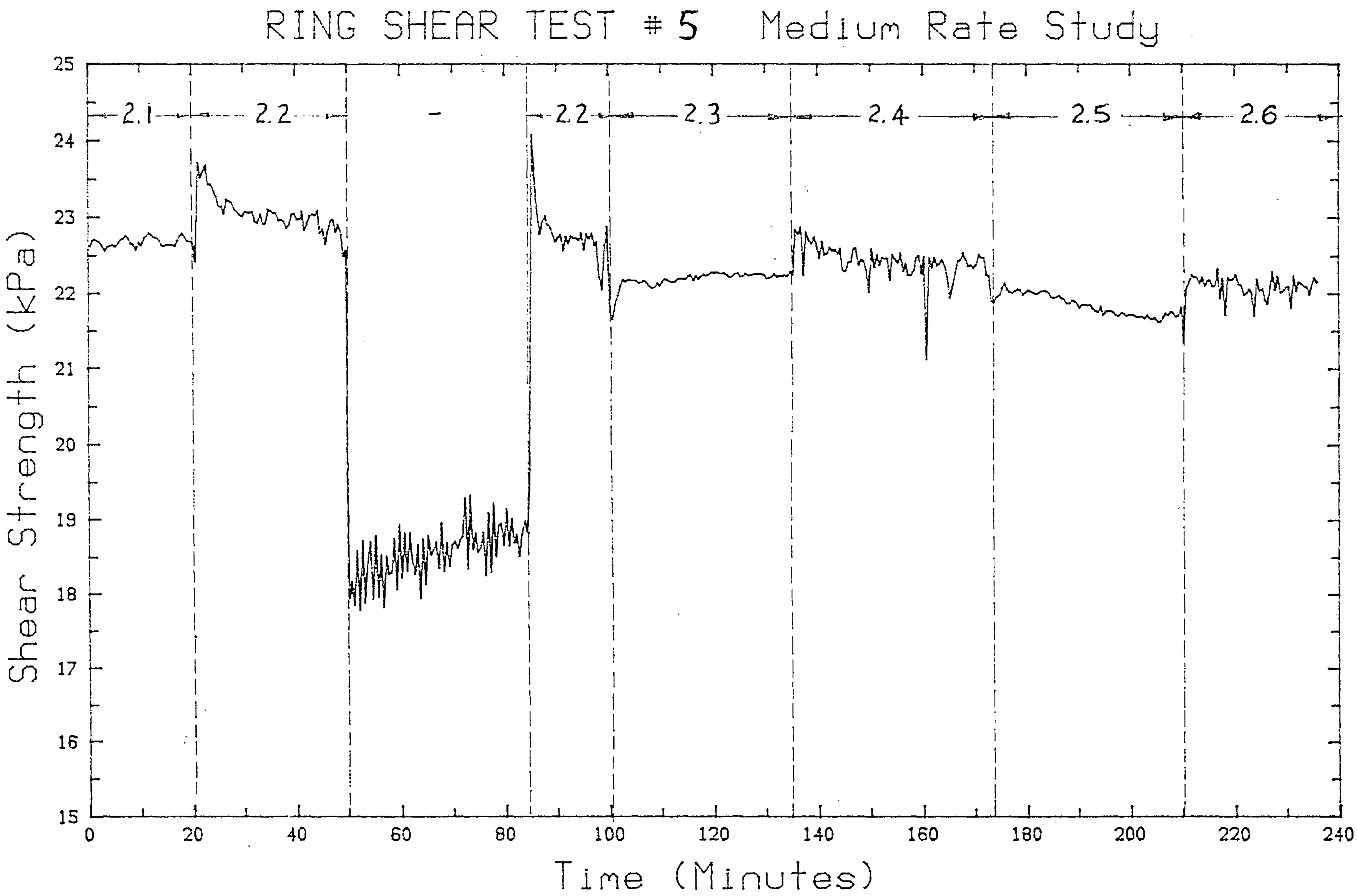
| Run No. | Shear Rate (mm/min) | Duration (min) | Δ Depth (mm) | Water Demand (ml) |
|---|---|---|---|---|
| 1.0 | #7 46.0 | 220 | 1.525 | 0.64 |
| 2.1 | #7 46.0 | 20 | 0.056 | 0.02 |
| 2.2 | #4t 0.124 | 20 | 0.000 | 0.00 |
| 2.3 | #6 7.0 | 35 | 0.000 | 0.06 |
| 2.4 | #4t 0.124 | 37 | 0.008 | 0.00 |
| 2.5 | #5 0.93 | 38 | 0.015 | 0.00 |
| 2.6 | #4t 0.124 | 23 | 0.000 | 0.00 |
| 3.1 | #4p 0.124 | 140 | 0.000 | 0.00 |
| 3.2 | #3 0.018 | 199 | 0.000 | 0.00 |
| 3.3 | #4p 0.124 | 49 | 0.000 | 0.00 |
| 3.4 | #1 0.00029 | 622 | 0.000 | 0.00 |
| 3.5 | #4p 0.124 | 118 | 0.004 | 0.00 |
| 3.6 | #2 0.0022 | 200 | 0.002 | 0.00 |
| 3.7 | #4p 0.124 | 60 | 0.000 | 0.00 |
| 4.1 | #4t 0.124 | 21 | 0.000 | 0.00 |
| 4.2 | #7 46.0 | 19 | 0.023 | 0.01 |
| 4.3 | #4t 0.124 | 20 | 0.022 | 0.00 |
| 4.4 | #8 310 | 20 | 0.160 | 0.21 |
| 4.5 | #4t 0.124 | 20 | 0.002 | -0.01 |
| 4.6 | #9 2079 | 20 | 2.529 | 1.24 |
| 4.7 | #4t 0.124 | 28 | 0.026 | -0.05 |

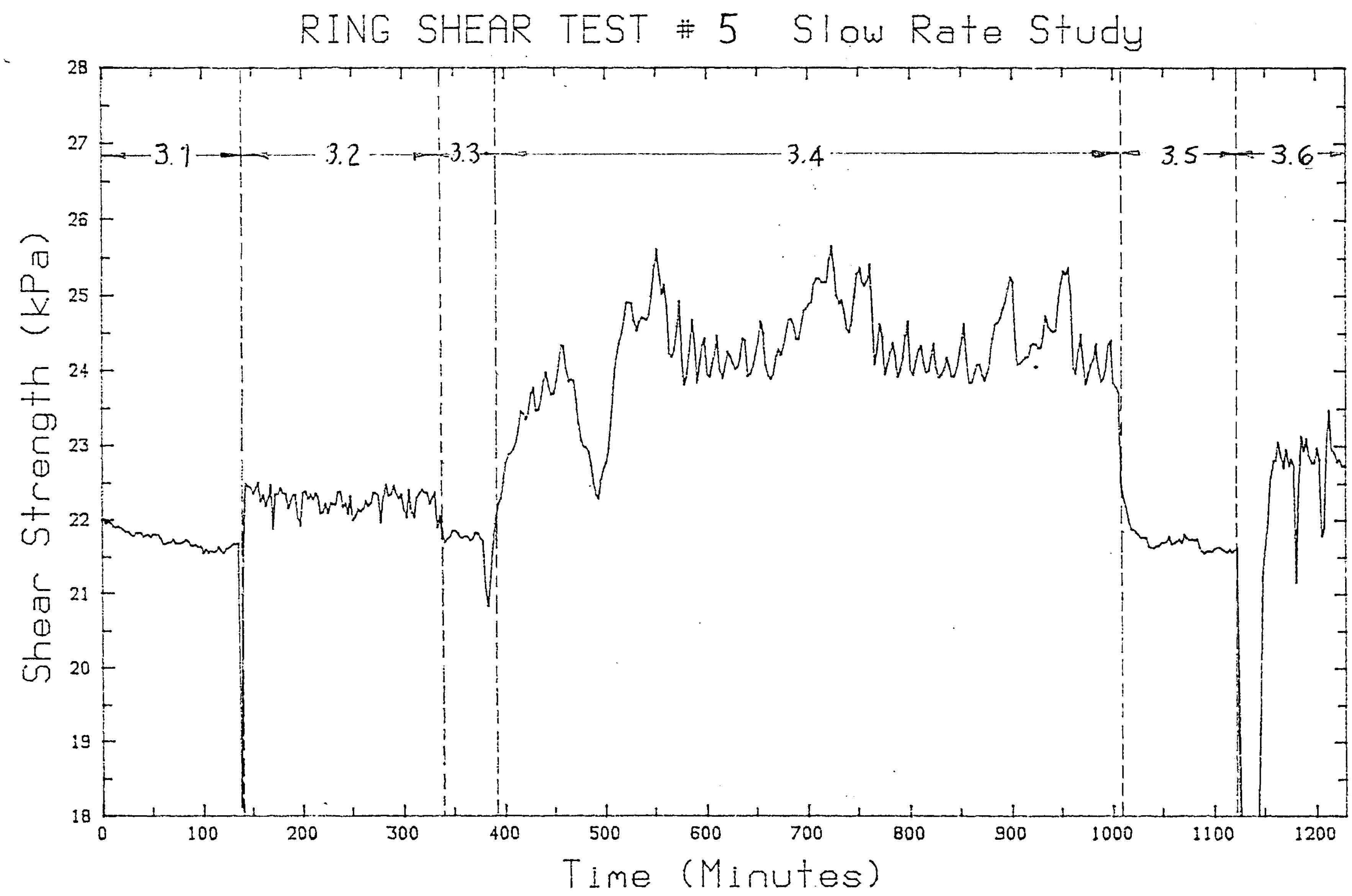
Table A4.5c ANALYSIS OF TEST #5 RESULTS

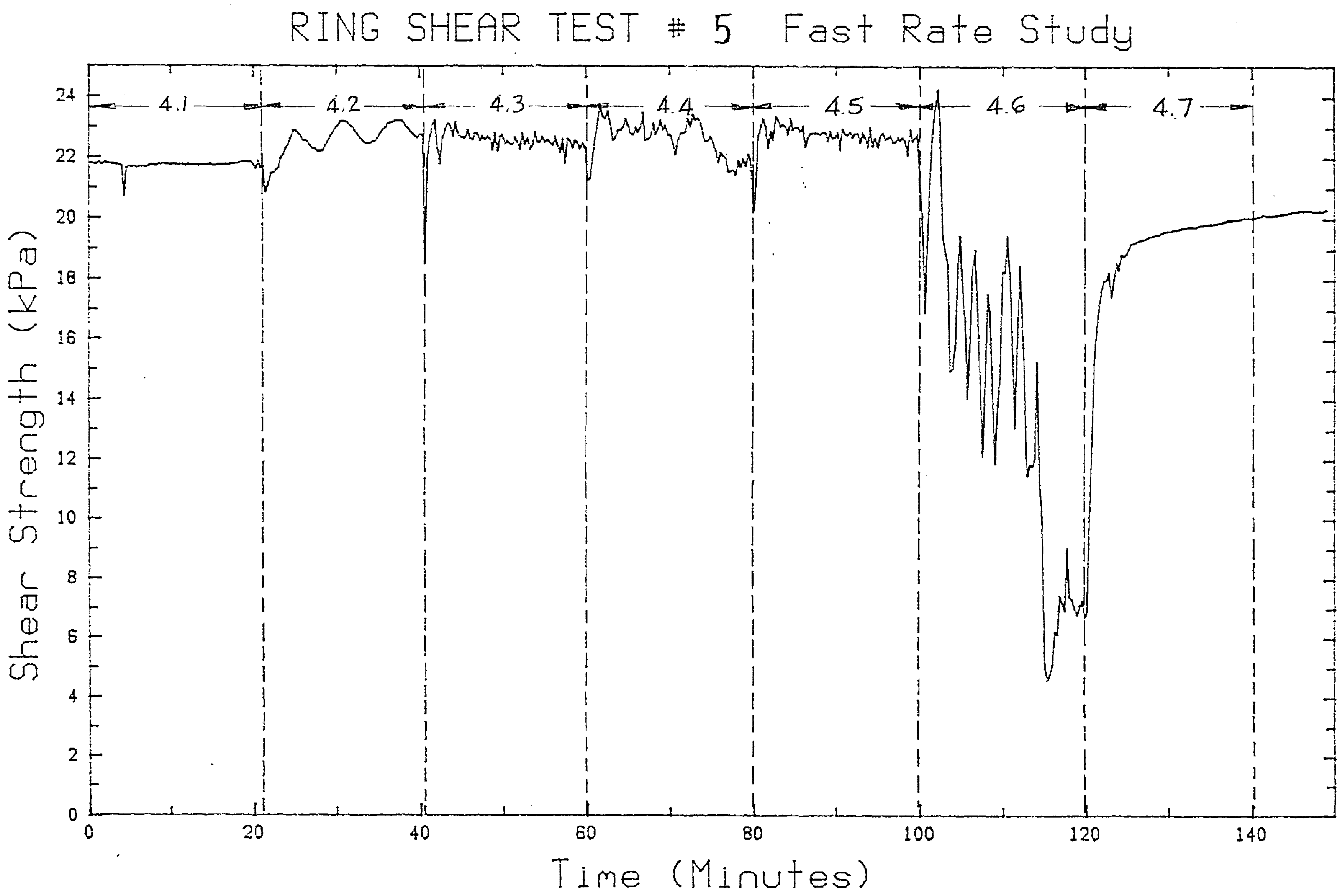
| Run No. | Shear Rate (mm/min) | Duration (min) | Average $\tau$ (kPa) | Stand. Dev. $\tau$ (kPa) |
|---|---|---|---|---|
| 1.0 | #7 46.0 | 220 | 22.630 | 0.122 |
| 2.1 | #7 46.0 | 20 | 22.677 | 0.085 |
| 2.2 | #4t 0.124 | 20 | 22.700 + | 0.585 + |
| 2.3 | #6 7.0 | 35 | 22.182 | 0.143 |
| 2.4 | #4t 0.124 | 37 | 22.424 | 0.595 |
| 2.5 | #5 0.93 | 38 | 21.851 | 0.200 |
| 2.6 | #4t 0.124 | 23 | 22.081 | 0.508 |
| 3.1 | #4p 0.124 | 140 | 21.734 | 0.170 |
| 3.2 | #3 0.018 | 199 | 22.202 | 0.744 |
| 3.3 | #4p 0.124 | 49 | 21.692 | 0.299 |
| 3.4 | #1 0.00029 | 622 | 24.430 + | 0.475 + |
| 3.5 | #4p 0.124 | 118 | 21.737 | 0.250 |
| 3.6 | #2 0.0022 | 200 | 22.235 + | 0.514 + |
| 3.7 | #4p 0.124 | 60 | * | * |
| 4.1 | #4t 0.124 | 21 | 21.721 | 0.247 |
| 4.2 | #7 46.0 | 19 | 22.587 | 0.518 |
| 4.3 | #4t 0.124 | 20 | 22.494 | 0.865 |
| 4.4 | #8 310 | 20 | 22.594 | 0.664 |
| 4.5 | #4t 0.124 | 20 | 22.691 | 0.743 |
| 4.6 | #9 2079 | 20 | ~ | ~ |
| 4.7 | #4t 0.124 | 28 | ~ | ~ |

\+ These statistics have been determined only over the range in which the strength had stabilised.
* The power supply to the data aquasition system failed during this test.
~ The strength never stabilised in these tests.


RING SHEAR TEST # 5 Rate 7 Normal Stress 50kPa
Shear Strength (kPa)
30
29
28
27
26
25
24
23
22
21
20
0
1
2
3
4
5
6
7
8
9
10
Displacement (m)


RING SHEAR TEST # 5 Medium Rate Study
Shear Strength (kPa)
Time (Minutes)
2.1
2.2
-
2.2
2.3
2.4
2.5
2.6
25
24
23
22
21
20
19
18
17
16
15
0
20
40
60
80
100
120
140
160
180
200
220
240


RING SHEAR TEST # 5 Slow Rate Study
3.1
3.2
3.3
3.4
3.5
3.6
Shear Strength (kPa)
28
27
26
25
24
23
22
21
20
19
18
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
Time (Minutes)


RING SHEAR TEST # 5 Fast Rate Study
4.1
4.2
4.3
4.4
4.5
4.6
4.7
Shear Strength (kPa)
0
2
4
6
8
10
12
14
16
18
20
22
24
Time (Minutes)
0
20
40
60
80
100
120
140